Supporting Documentation
Aroostook River
Fort Fairfield, Maine

Local Flood Protection

May, 1987

US Army Corps
of Engineers
New England Division

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A. PERTINENT DATA

1. Purpose

Local flood protection

2. Location

State - Maine
County - Aroostook
City - Fort Fairfield

3. Design Flood

Frequency - 100-year flood
Freeboard - 3 feet
D_{50} - 0.44 feet for 1 vertical to 2 horizontal slope

4. Dike

Type - Earth Fill with Stone Protection
Maximum height above streambank - 28 feet
Maximum height above landside toe - 18 feet
- 16 feet (alternate)
Slopes - Riverside - 1 vertical on 2.5 horizontal
- Landside - 1 vertical on 2.5 horizontal
Total Length - 3,175 feet
- 2,730 feet (alternate)
Top Width - 12 feet to 17 feet (transition sections)

5. Pump Station

Type - Concrete
Bottom Elevation - 342 feet NGVD
Capacity - 30 cubic feet per second

6. Pressure Conduit

Type - Concrete/Dustile Iron
Invert Elevation(s) - 366 feet NGVD to 342 feet NGVD
Diameter - 4 feet

7. Railroad Gates

Type - Stop log
Bottom of footing elevation - 354 feet NGVD

B. INTRODUCTION

8. Location and Description of Project

The proposed flood damage reduction project in Fort Fairfield, Maine is situated on the south bank of the Aroostook River. The Aroostook River originates approximately 61 miles to the southwest of Fort Fairfield at the east outlet of Munsungan Lake in Township 8, Range 9, Maine. It flows in a northeastly direction after passing through Fort Fairfield approximately 9 miles to its confluence with the St. John River in Four Falls, New Brunswick, Canada. The project will consist of a 3,175 or 2,730 foot (alternate) foot earth dike situated on the south bank of the Aroostook River, a pump station and pressure conduit to handle interior drainage, and two railroad gates to provide end closures for the dike. The project will reduce flood damage to private and commercial properties in the Fort Fairfield central business district during large flood events.

9. General

Subsurface investigations and geotechnical engineering studies were performed to further the continued planning of structural features to reduce flood damage in Fort Fairfield, Maine. The subsurface investigations included research of available information, geological studies, subsurface explorations and laboratory testing. The subsurface investigations were performed to determine the distribution and description of potential foundation materials for the proposed improvements. Preliminary geotechnical engineering studies, based on the data collected from the subsurface investigations were conducted to develop safe and economical preliminary foundation designs, dike sections, and construction methods.

Additional Plan Formulation was done after completion of subsurface investigations and most of the geotechnical engineering effort for this report. Changes due to the additional plan formulation are designated as "alternate" on the plates and in the text. Subsurface explorations and geotechnical studies will be required during the plans and specifications stage to accommodate the alternate pump station and south gate structure locations.

10. Elevations

All elevations mentioned in this report are in reference to the National Geodetic Vertical Datum (NGVD), which is the mean sea level of 1929.

C. TOPOGRAPHY, GEOLOGY AND SEISMICITY

11. Topography

The project site is on the south bank of the Aroostook River about nine river miles southwest from its confluence with the St. John River in New Brunswick, Canada. The centerline of the proposed dike is along a sloping river bank which averages about 80 feet wide and varies in elevation (El.) from approximately 340 feet to 360 feet. Terraces are well developed on the opposite bank of the river. Away from the river banks, low, rounded hills rise to about El. 700 feet.

12. Geology

The bedrock of the area is mapped as the Spragueville Formation, a calcareous metasiltstone with interbedded silty limestones. Borings along the alignment went to elevations as low as El. 327 feet with none reaching bedrock, but State of Maine Route 165 highway bridge borings reached bedrock as shallow as El. 345 feet where the railroad passes under the highway bridge just upstream of the project site. The borings show that the rock surface plunges as deep as El. 270 feet toward the north bank of the river, or about 75 feet below the water surface. Along the dike alignment the overburden consists of fill, sands and gravels with minor silts which overlie a sandy gravelly till.

13. Seismicity

The project is located in Seismic Zone 1 as defined by the map contained in Engineering Regulation, ER 1110-2-1806, "Earthquake Design and Analysis for Corps of Engineers Projects." A seismic coefficient of 0.05g is to be used for stability analyses of concrete structures.

D. SUBSURFACE INVESTIGATIONS

14. Presentation of Data

Locations of the subsurface explorations are shown on Plates B-1, B-2, and B-3. An Engineering-Log Profile of the borings is presented on Plate B-4. Probe data is shown on Table B-1. The results of soil tests are included in Table B-2.

15. Subsurface Explorations

Atlantic Testing Laboratories, Limited executed seven hollow stem auger borings (FD-86-7 to FD-86-13) for the United States Army Corps of Engineers (USACE), March 10-12, 1986. The boreholes were advanced in areas where proposed structures are to be constructed. Standard Penetration Tests and split spoon samples were generally taken at 5-foot intervals or more frequently when required by the inspector. The test borings were terminated at depths from 12 feet to 32 feet.

Three drive sample borings (FD-78-4 to FD-78-6) and six hand probes (FP-78-1 to FP-78-6) were performed and inspected by the USACE from September 27, 1978 to October 6, 1978. The borings were terminated at 25 feet of depth along the centerline of the proposed dike. Continuous sampling was performed in the boreholes by driving 2-1/2-inch and 2-inch inside diameter solid spoons with a 350 pound weight and 18-inch drop except where diamond core drilling was required to penetrate obstructions. The hand probes were advanced near the normal Aroostook River water line with an eight pound sledge to depths from 2.6 feet to 3.2 feet.

The Maine State Highway Commission performed 13 drive sample explorations for the Aroostook River bridge which is approximately 500 feet north (upstream) of the proposed dike. The explorations varied in depth from 20.0 feet to 88.8 feet. Solid tube samples were generally taken at 5-foot intervals. Rock was cored in 12 of the holes.

Three preliminary drive sample borings (FD-50-1 to FD-50-3) were executed and inspected by USACE, July 10-12, 1950. The borings were located along the existing Canadian Pacific Railroad main line track which is from 90 feet to 175 feet west (inland) of the proposed dike. The depth of the borings varied from 15.5 feet to 31.0 feet. Continuous sampling was performed in the boreholes by driving 2-1/2-inch and 2-inch inside diameter solid spoons with a 350 pound hammer and 18-inch drop.

16. Future Explorations

The south gate structure and pump house will be moved to the alternate locations shown on Plates B-1 and B-3. It is recommended that explorations be performed at their alternate locations during plans and specifications stage to identify the depth of firm undisturbed natural materials. It also is recommended that test pits be executed during plans and specifications stage to better define the extent of the rubble fill near FD-78-6, the soft clayey silts, sands, and gravels near FD-78-4, and the location of utility lines.

17. Laboratory Tests

All laboratory tests were performed in accordance with the procedures described in Corps of Engineers Manual EM 1110-2-1906, "Laboratory Soils Testing." All soil samples were visually classified in accordance with the Unified Soil Classification System. Grain size analyses, Atterberg Limit determinations, Hydrometer analyses, and Moisture Content determinations were performed on selected samples to help classify the materials encountered and to provide more precise data where required.

E. CHARACTERISTICS OF FOUNDATION MATERIALS

18. Dike

Most of the riverside toe of the dike will lie in the pool (normal water El. 352 feet) created by Tinker Dam which is located approximately one mile downstream. The six probes taken in the proposed toe area indicate that the depth to firm ground is 12 to 18 inches. The soil profile under the proposed dike, is granular fill underlain by silty sandy gravels (GM) and silty gravelly sands (SM). Exceptions to the profile were observed near FD-78-6 where rubble fill was encountered and FD-78-4 where clayey silts, sands, and gravels were observed beneath the fill.

The fill is a brown to dark brown, heterogeneous mixture of silt, sand and gravel with cinders, organic matter, brick fragments, porcelain fragments, glass, roots, concrete, tar paper, steel rods, cobbles, boulders and sometimes having an organic odor. The observed thickness of the fill varies from 1.5 feet to 22.0 feet. Blow counts recorded during standard penetration tests and solid spoon drives indicate the fill is very loose to very compact.

Light brown, brown and gray-brown silty sandy gravels (GM) and silty gravelly sands (SM) were observed below the fill. The silt content varied from 7 to 32 percent in grain size determinations performed on the silty sandy gravels and silty gravelly sands. Standard Penetration test and solid spoon sample blow counts indicate the silty sandy gravels and silty gravelly sands are very loose to very compact. Most of the very loose materials are near the top of the silty sandy gravel and silty gravelly sand layer.

19. Gate Structures and Pump Station

The soil profile beneath the gates structures and pump station is similar to the one beneath the dike. The fill thickness is 5.0 feet to 11.5 feet at the proposed north gate structure, 15.0 feet at the proposed pump station, and 0 feet to 5.0 feet at the proposed south gate structure.

20. Pressure Conduit

The soil profile along the proposed pressure conduit is granular fill underlain by a brown, sandy silt. The granular fill varies 0 feet to 4.0 feet in thickness and is similar to the fill material below the proposed dike embankment. The brown, sandy silt is nonplastic. It is loose to moderately compact in consistency based on standard penetration test results.

21. Groundwater

Groundwater was encountered in the boreholes from El. 348 feet to El. 351 feet except for FD-78-6 and FD-86-13 where none was observed, FD-86-11 (El. 343 feet), and FD-86-12 (El. 361 feet). It must be noted that fluctuations in the groundwater levels may occur because of variation in rainfall, snow, ice, temperature, or other factors which differ from the conditions present at the time the observations were made.

22. Shear Strength and Permeability

Shear strength and permeability tests were not performed on the foundation soils. The estimated angle of internal friction for the foundation soils is 28 to 30 degrees. The estimated coefficient of vertical permeability for the foundation soils is (0.3 to 3) x 10-4 cm/s. The estimates are based on visual examination of the samples, grain-size distribution curves, data from exploration logs and experience with similar materials.

23. Consolidation

Consolidation tests were not performed on samples of foundation soils. All soft and compressible surficial materials will be removed prior to the construction of the dike embankment. The consolidation characteristics and natural densities of the principally granular foundation soils beneath the surficial materials are such that significant post-construction foundation settlement is not anticipated under the proposed embankment loadings.

F. CHARACTERISTICS OF EMBANKMENT MATERIALS

24. General

Most of the materials from the required stripping and excavation operations will not be suitable for use in construction of the dike embankment. The suitable material from the excavation and stripping operations will be used to the extent practicable. The contractor will furnish all embankment materials other than those available from the required excavation and stripping operations due to the high cost of developing government furnished borrow areas and difficulty involved in acquiring the land for borrow areas.

25. Filter Design

The gradation requirements for impervious fill, gravel bedding, stone bedding, and stone protection have been established in accordance with the filter criteria set forth in Engineering Manual, EM 1110-2-1913, "Design and Construction of Levees."

26. Impervious Fill

Impervious fill will be furnished by the contractor. It will be a natural, reasonably well graded, unprocessed material which contains clay, silt, and sand. Experience with materials meeting the gradation ranges below indicates that placement moisture contents can be maintained within two percent of optimum moisture content with moderate control and that in-place dry densities will be approximately 135 pounds per cubic foot.

Sieve Size (U.S. Std.)	Percent Passing by Dry Weight
6-inch	100
3-inch	85-100
No. 4	70-95
No. 40	35-70
No. 200	20-45

27. Gravel Bedding

Gravel Bedding will be furnished by the contractor. It shall consist of tough, durable particles of sand and gravel or crushed stone which are reasonable well rounded. The materials shall be reasonably well graded within the limits specified below.

Sieve Size (U.S. Std.)	Percent Passing by Dry Weight
6-inch	100
1-inch	50-90
No. 4	25-75
No. 16	15-50
No. 200	0-5

(In addition, not more than 10 percent, by dry weight, of the component passing the No. 4 sieve shall pass the No. 200 sieve.)

28. Stone Bedding

Stone bedding will be furnished by the contractor. It shall consist of quarried rock, composed of hard, durable, angular and sound rock fragments. Stone bedding shall be reasonably well graded within the limits specified below.

Sieve Size (U.S. Std.)	Percent Passing by Dry Weight
6-inch	90-100
1-1/2 inch	0-40
No. 4	0-5

29. Stone Protection

Stone protection will be furnished by the contractor. It shall consist of quarried rock, composed of hard durable, angular and sound rock fragments with a unit weight of not less than 162 pounds per cubic foot. It shall meet the following gradation and size requirements.

Class	Limits of Stone Weight (Pounds)	Percent Lighter by Weight
I	Between 120 and 300 (Max)	100
	Between 60 and 90	50
	Less than 20	15
	2 (Min.)	0
II	Between 900 and 2300 (Max.)	100
	Between 450 and 700	50
	Less than 150	15
	2 (Min.)	0

30. Shear Strength and Permeability

It is estimated based on the above gradations that the proposed embankment materials will develop the following angles of internal friction and coefficients of permeability:

Materials	Angle of Internal Friction (Degrees)	Coefficient of Permeability (cm/s)
Compacted Impervious	30 + 32	$<10^{-4}$
Dumped Gravel	30 to 33	10^{-3} to 10^{-2}
Compacted Gravel	35 to 37	10^{-3} to 10^{-2}
Gravel Bedding	35 to 37	10^{-3} to 10^{-2}
Stone Bedding	40	$>10^{-2}$
Stone Protection	40	$>10^{-2}$

31. Sources

Sand, gravel, and stone could be supplied by a commercial supplier in Presque Isle which is approximately 10 miles from the proposed project site. Private sand, gravel, and stone sources exist along the Aroostook River within 5 miles of the project which have been opened for use on past projects. Concrete is available from the suppliers in Presque Isle, Houlton, and Madawaska which are all within 40 miles of the site.

G. DESIGN AND CONSTRUCTION

32. Design Criteria

The principles and procedures discussed in Engineering Manual, EM 1110-2-1913, "Design and Construction of Levees," were used to develop dike sections for this project. Layer thicknesses and stone sizes for the proposed stone protection on the dike were determined using procedures in the Engineering Manual, EM 1110-2-1601, "Hydraulic Design of Flood Control Channels" and Engineering Technical Letter, ER 1110-2-120, "Additional Guidance for Riprap Channel Protection."

33. Materials for Dike Construction

All dike materials will be furnished by the contractor. It is estimated that approximately 2,000 cubic yards of excavation will be required to remove unsatisfactory dike foundation materials. Most of the material excavated will not meet the specifications for the dike embankment materials. The Contractor will be required to dispose of the excavated material that can not be reused at an appropriate upland site.

34. Dike Sections

Proposed dike sections are shown on Plates B-5 and B-6. The shape of the sections was influenced by foundation conditions, seepage control requirements, river erosion, ice action, maintenance considerations, and construction sequence. The stone protection thickness is greater in Section A-A (Typical End Section) than the other sections to reduce erosion caused by eddy currents and ice action at the ends of the dike. The toe on the landside of the dike will interrupt seepage in critical areas and act as an inspection trench during construction. Stone will protect the dike from erosion and ice action on the riverside. Grass, placed at a 1 vertical on 2.5 horizontal slope for maintenance reasons, will protect the landside dike slope. The dumped gravel fill riverside berm will expedite construction of the dike and will allow the contractor to dewater the central dike base prior to placing the compacted impervious fill core. The compacted impervious fill core will cut off seepage.

35. Seepage Control

The design hydrostatic head for the dike is the difference between the 100-year flood level (El. 366 feet to El. 367 feet) on the waterside and a water level at the ground surface on the landside. The design hydrostatic head ranges from approximately 3 feet to approximately 15 feet. Seepage through the dike will be controlled by the relatively long seepage path through the impervious core. Foundation seepage will be controlled by the relatively long seepage path through the predominantly silty sandy gravel and silty gravelly sand foundation soils. A shallow landside toe drain will provided to interrupt seepage and reduce softening on the inside of the dike.

36. Embankment Stability

Section D-D was selected for stability analysis because it combines maximum embankment height with average to low foundation strengths. Section D-D was analyzed for stability against shear failure using circular failure surfaces and the UTEXAS2 slope stability package for the End of Construction, Sudden Drawdown from Maximum Pool, Intermediate Flood Stage, Steady Seepage from Maximum Pool conditions. An analysis of earthquake conditions was not judged necessary due to the height of the dike, the low magnitude of earthquakes that have occurred in the vicinity of the site in the past, and the characteristics of the dike materials. The design unit weights and shear strength parameters were selected on the basis of experience with similar materials on other projects and are tabulated below:

Material	Unit Weight (pcf)		Shear Strength (degrees, psf)		
	saturated	moist	Q	R	S
Stone Protection	135	118	40,0	40,0	40,0
Gravel Bedding and Compacted Gravel Fill	145	135	35,0	37,0	37,0
Dumped Gravel Fill	135	120	30,0	33,0	33,0
Compacted Impervious Fill	140	135	30,0	30,0	32,0
Foundation Soils (above El. 342.0 feet)	137	130	28,0	28,0	30,0
Foundation Soils (Below El. 342.0 feet)	140	133	30,0	30,0	32,0

The minimum factor of safety for each condition is shown below. The results indicate that the selected embankment is safe from shear failure.

Condition	Factor of Safety Acceptable	Calculated (Shallow)	Calculated (Deep)
End of Construction (Riverside)	1.3	1.6	1.5
End of Construction (Landside)	1.3	1.4	2.0
Sudden Drawdown from Maximum Pool (El. 367)	1.0	1.3	1.2
Intermediate Flood Stage (El. 360 and El. 356)	1.4	1.6	1.6
Steady Seepage from Maximum Pool (El. 367)	1.4	1.5	1.7

37. Dike Settlement

The embankment and foundation soils are of low compressibility except possibly for the rubble fill near FD-78-6 and the clayey silts, sands, and gravels near FD-78-4. The rubble fill and surficial, soft, clayey silts, sands and gravels will be removed prior to construction of the dike. The remaining clayey silts, sands and gravels are judged to be of low compressibilty in situ due to their high densities and low plasticity indices. Therefore, it is expected that all significant settlement of the principally granular embankment and foundation soils will occur during construction.

38. Construction Sequence

The dumped gravel fill riverside toe will be constructed starting at the upstream end by pushing material into and down the Aroostook River with bulldozers. The riverside toe will act as a cofferdam and will facilitate dewatering of the compacted fill areas by open pumping. Deleterious materials will be stripped in the compacted fill areas after completion of dewatering and prior to placement of fills. Compacted fills will be placed to their full width in reaches long enough to permit proper operation of compaction equipment. Stone protection and bedding layers will be placed below normal water without diversion or dewatering of the construction area immediately after completion of the dumped gravel fill riverside toe. Above normal water, they will be placed in the dry after completion of the compacted fills. Dike reaches will be completed to their full width including stone protection prior to flood season.

39. Placement and Compaction

Compacted gravel and impervious fill materials will be spread with bulldozers or other approved equipment in loose layers of 8 inches in non-restricted areas and 4 inches in restricted areas. Each layer will be compacted to 95 percent of its maximum dry unit weight as determined by modified proctor test ASTM D-1557. Heavy tractors and vibratory rollers will not be allowed in restricted areas.

40. Slope Protection

Hydraulic analysis for erosion control of the dike indicates that a minimum D_{50} stone size of 0.44 feet is adequate to resist tractive forces for a 1 vertical to 2 horizontal slope. A stone layer thickness of 0.75 feet was calculated from the minimum D_{50} stone size. The stone layer thickness was increased to 1.5 feet for placement above normal water to resist ice forces, and to 2.25 feet for placement below normal water to resist ice forces and to provide for uncertainties associated with underwater placement. The stone sizes required to construct layers 1.5 feet and 2.25 feet thick will be large enough to be considered vandal proof.

Experience with ice action at Fort Kent, Maine has shown embankment displacements occur in the transition areas even when twice the minimum D_{50} stone size is used to determine the layer thickness. Three times the minimum D_{50} stone size was used to calculate a stone layer thickness of 2.25 feet in the transition areas. The stone layer thickness in the transition areas were increased to 3.0 feet for placement above normal water to resist ice forces, and to 4.5 feet for placement below normal water to resist ice action and to provide for uncertainties associated with underwater placement.

The proposed classes and gradations for the stone protection are listed in Section 29. The proposed stone protection sections are shown on Plates B-5 and B-6.

41. Structures

A pump station, pressure conduit and two railroad gates will be appurtenant structures to the dike. They will be light weight structures constructed at the locations shown on Plates B-1 to B-3. They will be constructed on undisturbed natural soils or compacted gravel fill placed on undisturbed natural soils, and at least 6 feet below grade for adequate frost protection. The proposed bottom elevations for the structures are 354 feet for the gates, 348 feet for the pump station, and from 366 feet to 342 feet for the pressure conduit.

A design bearing pressure of 4000 pounds per square foot will be used to design the spread footings required for the gates and pump house. Design bearing pressures for footings less than three feet in minimum dimension will be reduced to B/3 times the recommended bearing pressure, where B is the smallest dimension of the footing in feet. A minimum width of 18 inches will be maintained for continuous footings.

42. Environmental

The environmental concerns identified to date are: movement of pesticides in the river bottom sediments during construction of the river side toe, disposal of stripped material and rubble fill, migration of fines downstream during the dewatering operation, and a petroleum odor in exploration FD-86-11. The results of an Impact Analysis Branch Sampling and Testing Program conducted during the winter and spring of 1986 indicate the levels of pesticides are not high enough in the river bottom sediments at the site to be concerned that significant amounts will move during construction. It is recommended that additional testing be performed during construction to insure pesticide movement is minimal. The town of Fort Fairfield and the state of Maine will identify appropriate disposal areas for the stripped material and rubble fill. Silt curtains or an alternative will be used to reduce migration of fines downstream during the dewatering operation. The downstream end of the dike will be moved to avoid possible contaminated materials in the vicinity of exploration FD-86-11.

43. Access

A gravel surface access road will run along the crown of the dike to allow for inspection, maintenance, recreation and flood-fighting activities. Either two access ramps and one turnout or one access ramp, one turnout, and turnaround will be provided to facilitate use of the access road. Locations for the access ramps, turnout, and turnaround will be decided during the plans and specifications stage.

44. Pipelines

One 16-inch sewer main and many smaller live and abandoned utility pipes exist under the proposed dike alinement. The sewer main and line utility lines will be moved outside the dike limits. The abandoned utility pipes will be removed prior to construction of the dike. The inspection trench and test pits will be used to search for lines that may not have been identified.

TABLE B-1

PROBES

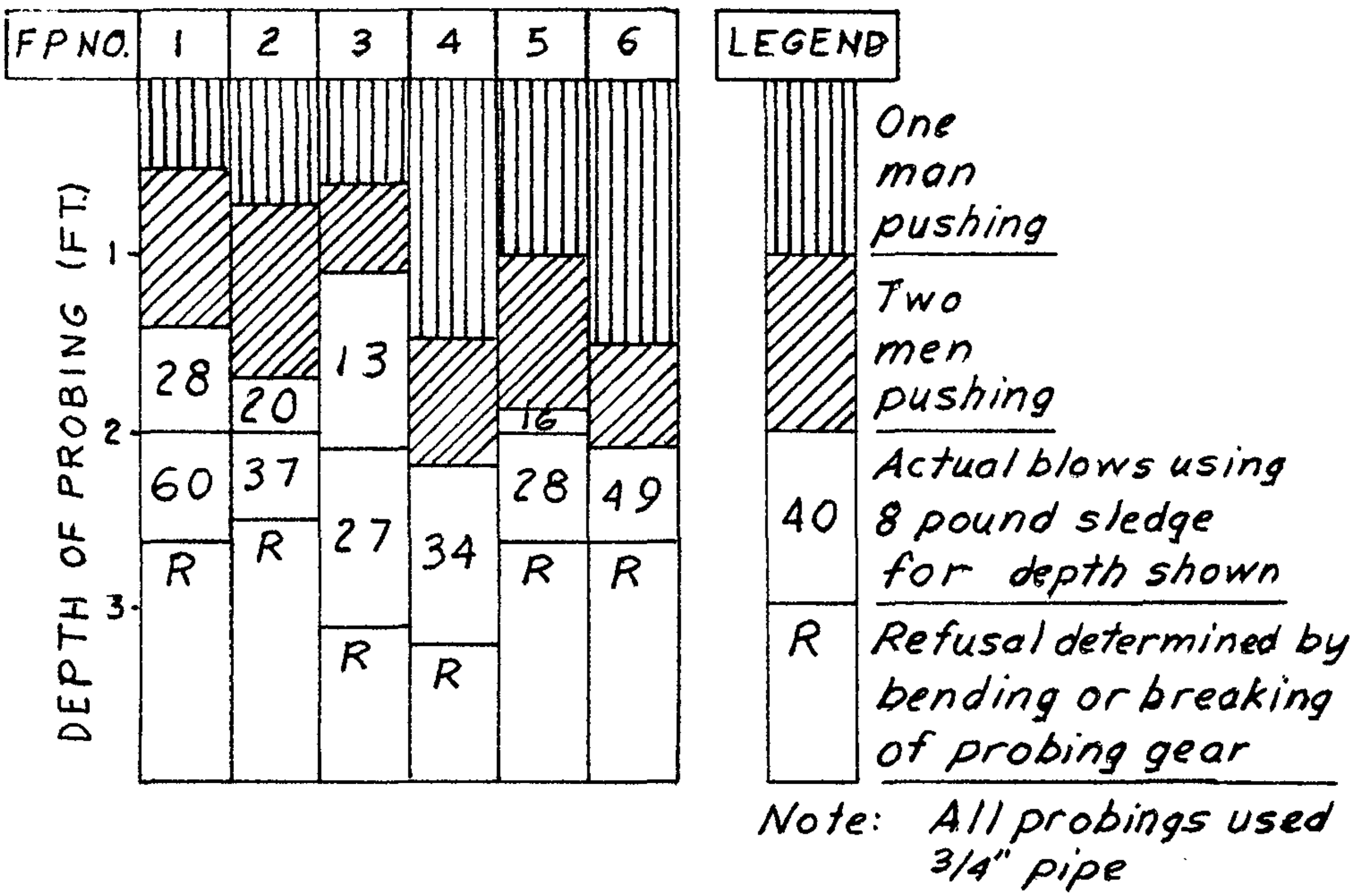

TABLE B-2

SOIL TESTS RESULTS

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	NAT. WATER CONTENT % DRY WT		COMPACTION DATA			NAT. DRY DENSITY LBS/CU FT		OTHER TESTS		
														STND. AASHO							
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL		TOTAL	- NO 4	OPT. WATER % DRY WT	MAX. DRY DENS. LBS/CU FT	* PVD LBS/CU FT	TOTAL	- NO 4	SHEAR	CONSOL.	PERM.
FD-78-4	352.0	J3	2.7-5.0	CL-ML	0	40	60	0.003	27	21		35.5									
FD-78-4	352.0	J7-1	6.7-10.2	CL	0	30	70	0.005	31	22		27.9									
FD-78-4	352.0	J7-2	6.7-10.2	CL	0	0	96	<0.001	27	19		22.2									
FD-78-4	352.0	J13	15.0-16.2	SC	5	65	30	0.015													
FD-78-4	352.0	J14	16.2-19.4	SC	38	40	22	0.025													
FD-78-4	352.0	J16	20.0-24.5	GC	55	30	15	0.03													
FD-78-5	351.0	J 7	5.6-10.0	GP-GM	59	34	7	0.3													
FD-86-7	361.0	S-3	10.0-12.0	SW-SM	24	69	7	0.2													
FD-86-8	361.0	S-3B	11.5-12.0	SM	1	67	32	-													
FD-86-8	361.0	S-4	15.0-17.0	SM	1	71	28	-													
FD-86-9	363.0	S-4	15.0-17.0	GM	47	40	13	-													
FD-86-10	361.0	S-2	5.0-7.0	SM	22	63	13	-													

* PROVIDENCE VIBRATED DENSITY TEST

SECTION C

STRUCTURAL DESIGN

DETAILED PROJECT REPORT
FORT FAIRFIELD LOCAL PROTECTION
FORT FAIRFIELD, MAINE

STRUCTURAL DESIGN REPORT

1. Purpose: The purpose of this report is to facilitate the review by a higher authority of the structural design features of the Fort Fairfield Local Protection Project, Fort Fairfield, Maine. This information is presented for inclusion in the Detailed Project Report, prepared under the special continuing authority of Section 205 of the 1948 Flood Control Act, as amended.

2. Introduction: This section presents the criteria, data, and assumptions used for the structural design of the proposed structu for this project. A brief description of each structure is provided and followed by stability computations designed to investigate the critical design condition.

3. Criteria Documents: Structural design criteria are contained in the publications listed below:

CORPS OF ENGINEER PUBLICATIONS

EC 1110-2-510	"Working draft of the Retaining and Flood Wall Manual", 31 August 1983 with Changes 15 July 1985.
ETL 1110-2-256	"Sliding Stability of Concrete Structures" 24 Jun 198
EM 1110-2-2501	"Flood Wall Manual" January 1948

4. MAJOR STRUCTURAL FEATURES:

The project involves (includes) two concrete stop-log structures, a pumping station and a 4-foot diameter Ductile Iron pressure conduit. The pressure conduit requires an inlet headwall, an outlet structure, and an emergency gate well. The gravity discharge conduit at the pumping station also requires a sluice gate well.

The following structures were analyzed for stability:

	Appendix A
a. Stop-log structure / Downstream	page 1
b. Stop-log structure / Upstream	page 9
c. Pressure-conduit Inlet headwall	page 15
d. Pumping station	page 19

Each set of stability computations provides a description of the structure, the design criteria and parameter values used in the calculations. The final design concepts of all the structural features of the job are presented in Plates (-) of this report.

5. <u>WORK TO BE COMPLETED</u>: During the preparation of plans and specifications, the detailing of all watertight joints, reinforcemen and equipment installation can be designed.

Special attention should be given to the design of the 48"Ø Ductile Iron pressure conduit. At any point under the main body of the dike, the conduit is under about a 29-foot head during an extreme flood condition. Pipe connections should be designed to withstand this pressure. An emergency gate well with a slide-type flap gate was designed to prevent excessive pressures resulting due to a river back flow condition. The flap gate would also allow for the interior drainage operation of the conduit during a flood conditio

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NEW ENGLAND DIVISION

CORPS OF ENGINEERS, U.S. ARMY

SUBJECT FORT FAIRFIELD - MAINE LPP

COMPUTATION STOP LOG - STRUCTURE / DOWNSTREAM

COMPUTED BY ENestorides CHECKED BY DATE 2-6-87

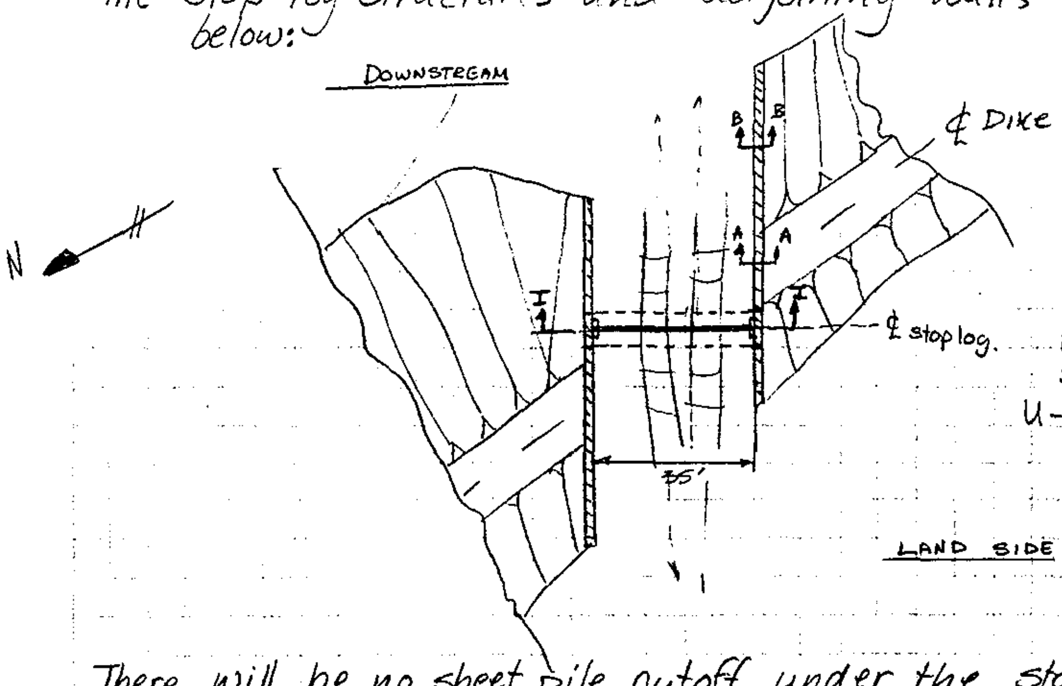
FORT FAIRFIELD - STOP LOG STRUCTURE / DOWNSTREAM

The stop log structure is a concrete U-channel with a center post and 2 bays of stop logs. The opening is about 35 feet wide and positioned on a skew with respect to the dike center line. On either end of the stop log structure concrete gravity walls extend to retain the dike embankment [See plan below].

The structures primarily function as a retaining wall and was designed in accordance to the following Corps Design Criteria:

1. EC 1110-2-510: "Working Draft of the Retaining and Flood Wall Manual" 31 August 1983 (w/changes 15 July '85)
2. EM 1110-2-2501: "Flood Wall Manual" January 1948 (w/changes
3. ETL 1110-2-256: "Sliding stability of Concrete Structures" 24 June

The stop log structures and adjoining walls are shown in plan below:



North wall ⇒ 74 ft long
South wall ⇒ 56 ft
U-Channel ⇒ 8 ft

There will be no sheet pile cutoff under the stop log structure nor a cut-off wall into the dike due to the low differential head and the long line of creep. $\Delta h = 5 ft$

Creep Ratio: along wall $\frac{57 ft}{5} \Rightarrow 11.4 > 4$ (permissible for sands gravels) OK ✓

under stoplogs $\frac{22}{5} \Rightarrow 4.4 > 4$ OK ✓

SUBJECT Fort Fairfield – Maine LPP
COMPUTATION Stop Log Structure / Downstream
COMPUTED BY ENestorides CHECKED BY DATE 2/6/87

Design Method

One section of the concrete U-channel and two typical sections of the gravity wall will be analyzed for stability under one load case. Load Case R2 was used for each section since it represents a rapid drawdown situation which is an extreme loading for retaining structure. Since the area in question is in seismic zone one (minor damage) the earthquake condition is not critical.

All the structures are founded on compacted gravel fill and under Load Case R2, they must satisfy the following stability criteria:

a) 75% of the base must be in compression,
b) the factor of safety against sliding must be greater than 1.33,
and c) bearing pressures should not exceed the allowable $4^{k}/_{ft^2}$.

Design & Soil Parameters:

	γ_{moist} (kcf)	$\gamma_{saturated}$ (kcf)	$\gamma_{submerged}$ (kcf)	ϕ	assume c / δ
Dumped Gravel bedding (RR)	.120	.135	.073	32°	0 / 0°
Compacted Impervious	.135	.140	.078	30°	0/0°
foundation soil	.130	.137	.075	28°	0/0°

- Allowable bearing capacity for foundation $2^{T}/_{ft^2} \Rightarrow 4^{k}/_{ft^2}$ [GEB]
- Min. Frost Penetration depth required ⇒ 6 feet
- Consider Soil pressures using K_o (at rest)

$$K_o = 1 - \sin\phi \text{ (Jaky's formula)} \qquad K_o = 1 - .5 \Rightarrow \boxed{K_o = .5}$$
$$\phi = 30^\circ$$

- for sliding:

$$\mu = \tan\phi \Rightarrow \mu = .53 \qquad \phi = 28^\circ \quad c = 0$$

- γ_w = 62.5 pcf.
- Rapid Drawdown Condition: This condition was modeled by treating 60% of the unbalanced fill as submerged and the remaining 40% as saturated.

Design Sections:

The design sections will be on separate plates accompaning this report.

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PAGE 3

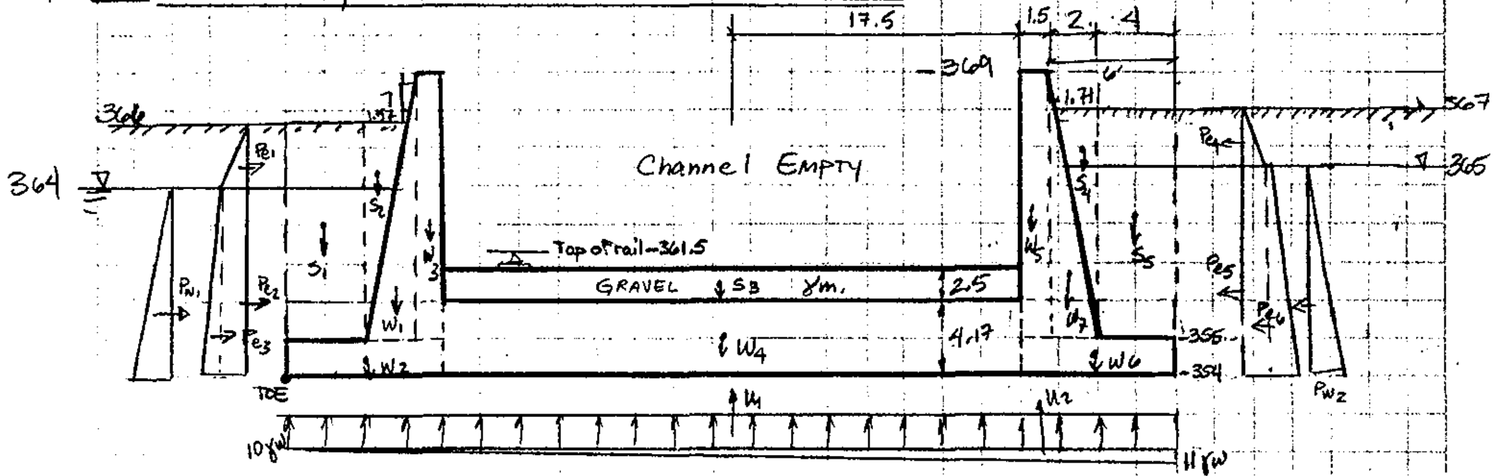
SUBJECT Fort Fairfield - Maine LPP

COMPUTATION Stop Log Structure / Downstream

COMPUTED BY ENestorides CHECKED BY DATE 2/6-87

I. Stop Log Structure → Section I-I:

Not to Scale



Waterlevel was determined by assuming 60% of unbalanced fill as submerge. and remaining 40% saturated. Ground level is at 361.

Item	COMPACTION	VERTICAL FORCES	Horizontal Forces	Moment arm	Moment
W_1	$\frac{1}{2}(14.0)(2.0)(.15)$	2.10		5.33	11.19
W_2	$(7.5)(1.0)(.15)$	1.13		3.75	4.24
W_3	$(1.5)(14.0)(.15)$	3.15		6.75	21.26
W_4	$(4.17)(35)(.15)$	21.89		25	547.31
W_5	$(1.5)(14.0)(.15)$	3.15		43.25	136.24
W_6	$(7.5)(1.0)(.15)$	1.13		46.25	52.26
W_7	$\frac{1}{2}(14.0)(2.0)(.15)$	2.10		44.67	93.80
S_1	$(11.0)(4.0)(.14)$	6.16		2.0	12.32
S_2	$\frac{1}{2}(11.0)(1.57)(.14)$	1.21		4.52	5.47
S_3	$(2.5)(35)(.12)$	10.5		25.0	262.50
S_4	$\frac{1}{2}(12.0)(1.71)(.14)$	1.44		45.43	65.42
S_5	$(4.0)(12.0)(.14)$	6.72		48.0	322.56
U_1	$-10(.0625)(50)$	−31.25		25.0	−781.25
U_2	$-\frac{1}{2}(.0625)(1)(50)$	−1.56		33.33	−52.08
Pe_1	$\frac{1}{2}(.5)(.14)(2)^2$		.14	10.67	1.49
Pe_2	$(.5)(.14)(2)(10)$		1.4	5.0	7.00
Pe_3	$\frac{1}{2}(.5)(.078)(10)^2$		1.95	3.3	6.44
Pe_4	$-\frac{1}{2}(.5)(.14)(2)^2$		−.14	11.67	−1.63
Pe_5	$-(.5)(.14)(2)(11)$		−1.54	5.5	−8.47
Pe_6	$-\frac{1}{2}(.5)(.078)(11)^2$		−2.36	3.67	−8.66
Pw_1	$\frac{1}{2}(.0625)(10)^2$		3.13	3.3	10.33
Pw_2	$-\frac{1}{2}(.0625)(11)^2$		−3.78	3.67	−13.87
Σ	—	ΣV = 27.87	ΣH = −1.20	—	ΣM = 693.87

SUBJECT: FORT FAIRFIELD - MAINE LPP

COMPUTATION: STOP LOG STRUCTURE / DOWNSTREAM

COMPUTED BY: ENestorides CHECKED BY: ______ DATE: 2-6-87

Stability at Toe el. 354:

- Overturning: $\frac{\Sigma M}{\Sigma V} = \frac{693.87}{27.87} = 24.9$ within mid $\frac{1}{3}$

  100% in bearing > 75% ok

- Sliding: $SF = \frac{\Sigma V \tan\phi + cl^{\to 0}}{\Sigma H} = \frac{27.87(.53)}{1.2} \Rightarrow 12.31 > 1.33$ OK ✓

- Bearing: $f_{\pm} = \frac{\Sigma V}{area} \pm \frac{Mc}{I}$ $I = \frac{(1)(50)^3}{12} = 10{,}416.7$

  $f_{\pm} = \frac{27.82}{50} \pm \frac{27.82(0.1)(25)}{10{,}416.7} \Rightarrow .56 \pm .01 \Rightarrow f_+ = .57\ ^{k}/_{ft^2}$

  $f_- = .55\ ^{k}/_{ft^2}$ $\Big\} < 4^{k}/_{ft^2}$ ok

Stability ⊥ to the stoplog centerline:

Flood level ⇒ 366.0
Ground level behind stop logs. ⇒ 361 } 5ft diffential head.

$P_W = \frac{1}{2}(.0625)(5)^2$
⇒ 27.84 k

Weight of structural wedge (including uplift.) ⇒ $27.87\ ^{K}/_{ft_{strip}} \times 8$ ft wide
222.96 K

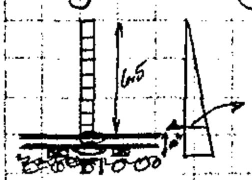
Sliding safety factor ⇒ $\frac{\Sigma V \tan\phi + cl^{\to 0}}{\Sigma H} \Rightarrow \frac{(222.96)(\tan 28)}{27.34} = 4.34 > 1.33$

OK ✓

Therefore, the U-channel structure satisfies criteria for overturning, sliding, and bearing pressure.

Stop-logs: The opening will be sand bagged around the rails and logs will start at el. 361.5 (top. of rail). The 100 yr. event is 366.

Log wall height required ⇒ 366.0 − 361.5 ⇒ 4.5 feet + 2 ft freeboard.
6.5 ft of logs.



Max. Pressure at bottom log: $6.5\,\gamma_w \Rightarrow 406.25\ ^{lb}/_{ft^2}$

say timber is 10"x10" ⇒ 9½" dressed ⇒ $406.25\ ^{lb}/_{ft^2} \times \frac{9.5}{12} = 321.61\ ^{lb}/_{ft}$

$M_{max} = \frac{wl^2}{8} = \frac{321.61(17.5)^2}{8} = 12{,}311.8$ lb·ft

$S_{10x10} = 142.896\ in^3$ $\sigma = M/S = \frac{(12311.8)(12)}{142.896} \Rightarrow 1033.91\ ^{lb}/_{in^2}$

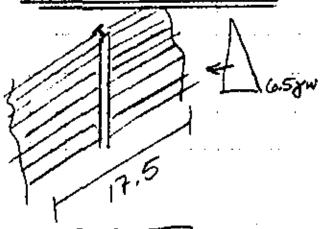
Ave. allowable bending stress (Oak, white, red) ~ 1200 psi OK ✓
extreme fiber

$\boxed{\text{Use 2 bays of 9 − 10"×10" timber logs 17.5 feet long.}}$

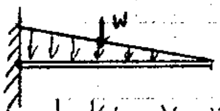
SUBJECT: FORT FAIRFIELD – MAINE LPP
COMPUTATION: Stop Log STRUCTURE / DOWNSTREAM
COMPUTED BY: ENestorides CHECKED BY: DATE: 2-6-87

Center Post:

Max load $\Rightarrow (6.5)(17.5)(.0625\ \text{kcf}) = 7.11\ \text{k/ft}$



Since the center post will be fixed at the base, it will act as a cantilever under a triangular load.



$M_{max} = \frac{Wl}{3} = \frac{(23.11)(6.5)}{3} \Rightarrow \underline{50.06\ \text{k.ft}}$

A36 steel
$f_b = .66 F_y = 23.76\ \text{ksi}$

$W = \frac{1}{2}(1.094)(6.5)^2$
$W = 23.11$

$V_{shear_{max}} = W = \underline{23.11\ \text{k}}$

$S_{req.} = \frac{M_{max}}{f_{bend.}} \Rightarrow \frac{50.06 \times 12^{\text{in}}/_{\text{ft}}}{23.76} = 25.28\ \text{in}^3$ try W12x26 $S = 33.4\ \text{in}^3$
W12x22 $S = 25.4\ \text{in}^3$

Since the W-section must fit the 9½" stop log, a W12 section is needed.

$\boxed{\text{Use W12x26}}$ (larger flange.)

Deflection at free end $\delta = \frac{Wl^3}{15EI}$ $E = 29{,}000$ $I = 204\ \text{in}^4$

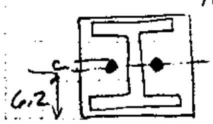
$\delta = \frac{(23.11)(6.5 \times 12)^3}{15(29000)(204)} \Rightarrow \underline{\delta_{max} = .123\ \text{in} \sim 1/8"}$

Brace not required however anchorage is important.

Anchorage:

① Anchor bolts: Assume A325 bolts. Shear at base = 23.11 k

try 2 7/8"φ anchor bolts – $f_v = \frac{23.11}{2(.601)} = 19.26\ \text{ksi}$
Area = .601 in²



$I = \Sigma Ad^2$
$I = 2(.601)(6.2)^2$
$I = 45.39\ \text{in}^4$

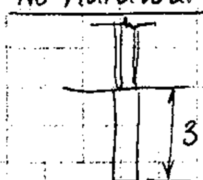
– $f_t = \frac{Mc}{I} \Rightarrow \frac{\overset{M}{(23.11)(2.17 \times 12)}(.44)}{45.39} = 5.79\ \text{ksi}$

– Allowables: $F_v = 21\ \text{ksi}$

for combined stress $F_t = 55 - 1.8 f_v \leq 44$
$F_t = 55 - 1.8(19.26)$
$= \underline{\underline{20.33}} \leq 44$ $F_t = 20.33\ \text{ksi}$

$f_{t\,actual} = 5.79 < 20.33$ $\underline{\underline{OK}}$ ✓
$f_{v\,actual} = 19.26 < 21$ OK ✓

② No hardware: Fit W12x26 into a 7"x12" – 3 foot long depression



This should be adequate anchorage and the depression can be formed in the U-channel base slab.

NED FORM 223
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NEW ENGLAND DIVISION

CORPS OF ENGINEERS, U.S. ARMY

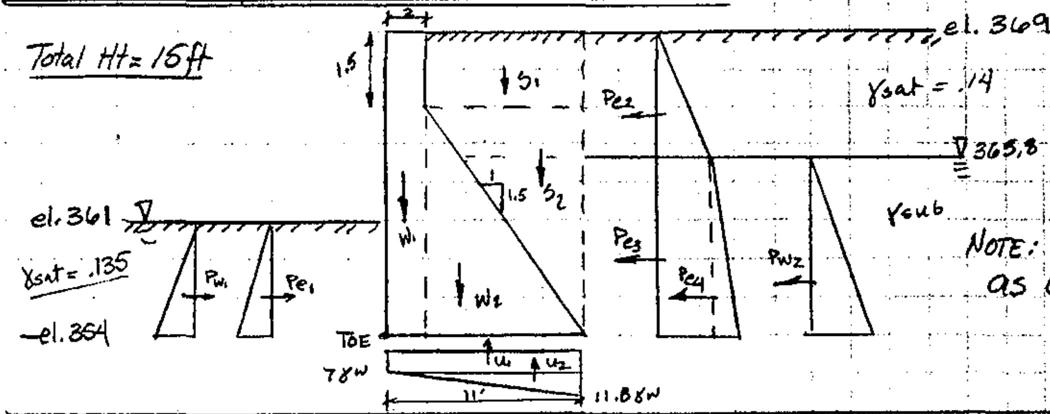
SUBJECT FORT FAIRFIELD - MAINE LPP

COMPUTATION Gravity Walls / DOWNSTREAM

COMPUTED BY E.Nestorides CHECKED BY ______ DATE 2-7-87

II GRAVITY WALL — SECTION A-A:

Total Ht = 15ft



NOTE: Water level is determi... as 60% of unbalanced f...

Item	COMPUTATION	Vertical forces	Horizontal Forces	Moment arm	MOMENT (+)
W_1	$2(15)(.15)$	4.5		1.0	4.5
W_2	$\frac{1}{2}(13.5)(9)(.15)$	9.11		5.0	45.55
S_1	$(1.5)(9)(.14)$	1.89		6.5	10.73
S_2	$\frac{1}{2}(13.5)(9)(.14)$	8.51		8.0	68.08
U_1	$-7(.0625)11$	- 4.81		5.5	- 26.46
U_2	$-\frac{1}{2}(.0625)(4.8)(11)$	- 1.65		7.33	- 12.1
P_{W1}	$\frac{1}{2}(.0625)(7)^2$		1.53	2.33	3.57
P_{W2}	$-\frac{1}{2}(.0625)(11.8)^2$		- 4.35	3.93	- 17.11
P_{e1}	$\frac{1}{2}(.5)(.135)(7)^2$		1.65	2.33	3.85
P_{e2}	$-\frac{1}{2}(.5)(.14)(3.2)^2$		-.36	12.87	- 4.63
P_{e3}	$-(.5)(.14)(3.2)(11.8)$		- 2.64	5.9	- 15.58
P_{e4}	$-\frac{1}{2}(.5)(.078)(11.8)^2$		- 2.72	3.93	- 10.70
Σ	—	ΣV = 17.55	ΣH = -6.89	—	ΣM = 49.7

Stability at Toe

- Overturning: $\frac{\Sigma M}{\Sigma V} = \frac{49.7}{17.55} \Rightarrow 2.83$ % in bearing $= \frac{3(2.83)}{11} \times 100 \Rightarrow 77.2\% > 75$ OK

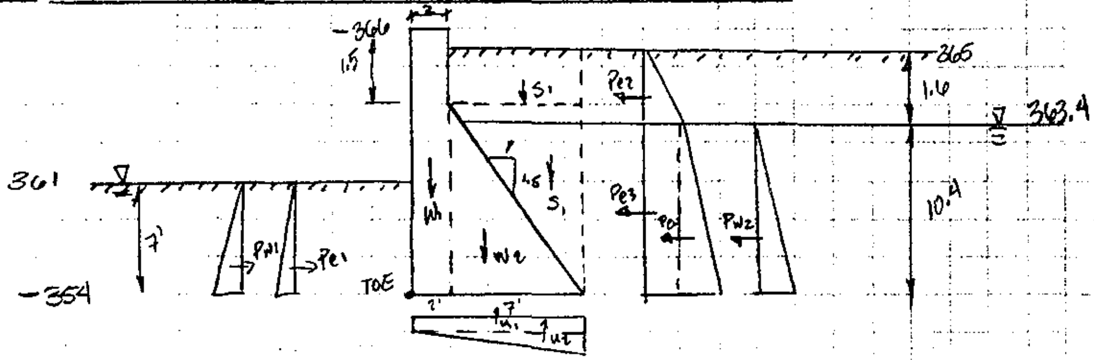
- Sliding: $SF = \frac{\Sigma V \tan\phi + cL^{\to 0}}{\Sigma H} = \frac{(17.55)\tan 28^\circ}{6.89} \Rightarrow 1.35 > 1.33$ OK

- Bearing Pressure: $f_{max} = \frac{2\Sigma V}{3a} \Rightarrow \frac{2(17.55)}{3(2.83)} \Rightarrow 4.13\ k/ft^2 \sim 4k/ft^2$

Close enough for an extreme load condition. OK

SUBJECT Fort Fairfield - Maine LPP
COMPUTATION Gravity Walls / Downstream
COMPUTED BY E Nestorides CHECKED BY DATE 2-7-87

III. Gravity Wall - Section B-B:



Item	COMPUTATION	Vertical forces	Horizontal forces	Moment arm	Moment
W_1	$2(12)(.15)$	3.6		1.0	3.6
W_2	$\frac{1}{2}(10.5)(7)(.15)$	5.51		4.33	23.86
S_1	$1.6(7)(.14)$	1.57		5.5	8.64
S_2	$\frac{1}{2}(10.5)(7)(.14)$	5.15		6.67	34.35
U_1	$-7(9)(.0625)$	-3.94		4.5	-17.73
U_2	$-\frac{1}{2}(3.4)(9)(.0625)$	$-.96$		6.0	-5.76
P_{w1}	$\frac{1}{2}(.0625)(7)^2$		1.53	2.33	3.57
P_{w2}	$-\frac{1}{2}(.0625)(10.4)^2$		-3.38	3.47	-11.73
P_{e1}	$\frac{1}{2}(.5)(.135)(7)^2$		1.65	2.33	3.85
P_{e2}	$-\frac{1}{2}(.5)(.14)(1.6)^2$		$-.09$	10.93	$-.98$
P_{e3}	$-(.5)(.14)(1.6)(10.4)$		-1.16	5.2	-6.03
P_{e4}	$-\frac{1}{2}(.5)(.078)(10.4)^2$		-2.11	3.47	-7.32
Σ	—	$\Sigma V = 10.93$	$\Sigma H = -3.56$	—	$\Sigma M = 28.32$

Stability at Toe

- Overturning: $\frac{\Sigma M}{\Sigma V} = \frac{28.32}{10.93} = 2.59$ % in bearing $= \frac{3(2.59)}{9} \times 100\% = 86.4\% > 75$ OK ✓

- Sliding: $SF = \frac{\Sigma V \tan 28^\circ}{\Sigma H} = \frac{10.93(.53)}{3.56} \Rightarrow 1.63 > 1.33$ OK ✓

- Bearing: $f_{max} = \frac{2\Sigma V}{3a} \Rightarrow \frac{2(10.93)}{3(2.59)} \Rightarrow 2.81 \ k/ft^2 < 4 k/ft^2$ OK ✓

SUBJECT: FORT FAIRFIELD - MAINE LPP

COMPUTATION: STOP-LOG STRUCTURE / DOWNSTREAM

COMPUTED BY: ENestorides CHECKED BY: DATE: 2-18-87

Logs:

Change to 27' opening } 13.5' per log

From previous calculations: Max pressure at bottom log $\Rightarrow$ 406.25 lb/ft²

if timber is 8"x8" $\Rightarrow$ 7½" dressed } $406.25 \times \frac{7.5}{12} = 253.91$ lb/ft

$M_{max} = \frac{wl^2}{8} = \frac{(253.91)(13.5)^2}{8} = 5784.3$ ft·lb

$S_{8\times8} = 70.313\ in^3$ $\sigma = \frac{M}{S} = \frac{(5784.3)(12''/1)}{70.313} = 987.18$ psi

allowable for white oak ~ 1200 psi OK.

Try 6"x 6" $\Rightarrow$ 5½ dressed $406.25 \times \frac{5.5}{12} \Rightarrow 186.20$ lb/ft

$M_{max} = \frac{(186.20)(13.5)^2}{8} = 4241.82$ ft.lb

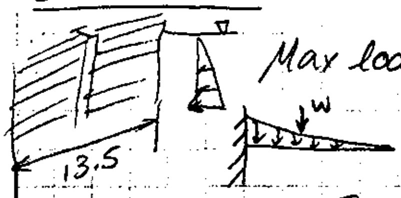
$S_{6\times6} = 27.729\ in^3$ $\sigma = \frac{M}{S} = \frac{4241.82 \times 12}{27.729} \Rightarrow 1835.69$ psi

NOT OK

USE 8" x 8" $\Rightarrow$ 2 bays, each 13.5 ft long

Total: 22 logs required

Center Post:



Max load $\Rightarrow 6.5(\gamma w)(13.5) \Rightarrow 5.48$ k/ft $W = \frac{1}{2}(\gamma w)(13.5)(6.5)^2$

$W = 17.82$

$M_{max} = \frac{Wl}{3} = \frac{17.82(6.5)}{3} = 38.62$ k.ft

$f_b = 23.76$ ksi $= .66 F_y$

$S_{req} = \frac{M_{max}}{f_{bend}} = \frac{(38.62 \times 12''/1)}{23.76} = 19.5\ in^3$

Use W10x22 $S_x = 23.2\ in^3$

$b_f = 5.75$ in. $k_1 = \frac{1}{2}$

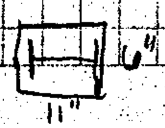
Length of bearing $= \frac{b_f}{2} - k_1 \Rightarrow 2.375$ in OK

STOP GROOVE:

8" groove

Center Post anchorage:

3-ft deep depression 6" x 11"



NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT: FORT FAIRFIELD - MAINE LPP.
COMPUTATION: STOP-LOG STRUCTURE - UPSTREAM
COMPUTED BY: ENestorides CHECKED BY: DATE: 2/5/87

FORT FAIRFIELD : STOP LOG STRUCTURE / UPSTREAM

The stop-log structure is a U-channel with one bay of stop-logs. The southern wall will abut against an existing concrete retaining wall while the north wall will retain the dike embankment. The north wall will also extend past the channel section to include concrete retaining walls on either end.

The structures were designed in accordance to the following Corps Criteria:

1. EC 1110-2-510 "Working Draft of the Retaining and Flood wall Manual" 31 Aug. 1983 w/revisions 15 July 1985.
2. ETL 1110-2-256 "Sliding stability for Concrete structures." 24 June 1981.
3. EM 1110-2-2501 "Flood Wall Manual" Jan. 1948 w/Changes.

LENGTH OF CHANNEL:

The use of sheetpile cutoff is discouraged due to the hardness of the foundation material. Therefore, a U-channel length was chosen [based on the differental head occuring during a flood] to minimize the possibility of seepage under the channel.

EM 1110-2-2501 Flood Wall Manual Para. 108(c)

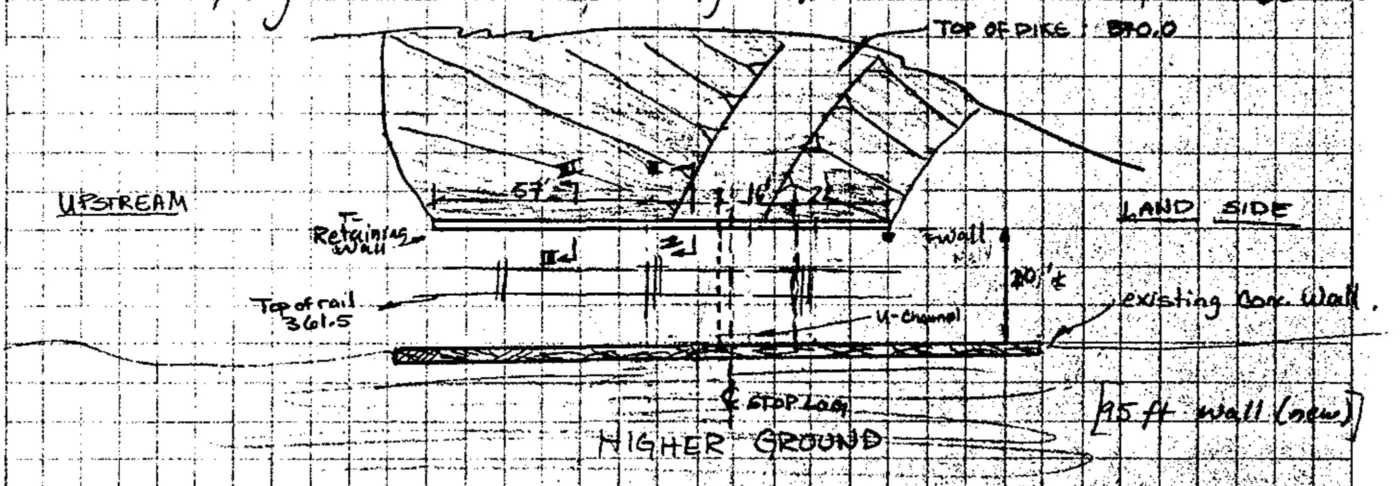
Creep Ratio $\Rightarrow \dfrac{\text{Line of Creep}}{\Delta \text{ Head}}$ if ratio ≥ 4 (for sands, gravels no additional cutoff needed)

Δ Head $\Rightarrow$ (flood level) − (tail-ground level) $\Rightarrow$ (367.5 − 360.0) $\Rightarrow$ 7.5 ft

4×7.5 = min, line of creep $\Rightarrow$ 30.0 use a 16.0 ft. long section.

Frost depth 6 feet min.

The stop log structure and retaining walls are shown in plan below:



Each structure will be analysed separately in the pages to follow.

NED FORM 223
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NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT: FORT FAIRFIELD – MAINE – LPP
COMPUTATION: STOP LOG STRUCTURE – UPSTREAM
COMPUTED BY: E Nestorides CHECKED BY: DATE: 2-5-87

DESIGN METHOD:

One section of the concrete U-channel and two typical sections of the retaining wall will be analyzed for stability under one Load case: LOAD case R2 as described in the draft of the Retaining and flood Wall Manual.

This load case represents a rapid drawdown situation which is an extreme loading for retaining structure. Since the area in question is in seismic zone One (minor damage) the earthquake condition is not critical.

All the structures are founded on compacted gravel fill or in certain areas, on the naturally deposited soils. Under Load Case R2, the structures must satisfy the following stability criteria:

a) 75% of the base must be in compression,
b) the factor of safety against sliding must be greater than 1.33,
c) bearing pressures should not exceed the allowable $4\,k/ft^2$.

Design and Soil Parameters:

	γ_{moist}	$\gamma_{saturated}$	$\gamma_{submerged}$	ϕ	c / δ
Dumped Gravel bedding (RR)	.120	.135	.073	32°	0 / 0°
Compacted Impervious	.135	.140	.078	30°	0/0°
foundation soil.	.130	.137	.075	28°	0/0°

– Consider Soil pressures using $K_o \Rightarrow$ at rest coefficient.

$K_o = 1 - \sin\phi \quad \phi = 30° \quad \boxed{K_o = .5}$

– For Sliding: $\mu = \tan\phi \quad \phi = 28°, \; c = 0\ tsf \quad \boxed{\mu = .53}$

– RAPID DRAWDOWN: This condition was modeled by treating 60% of the unbalanced fill as submerged and the remaining 40% as saturated.

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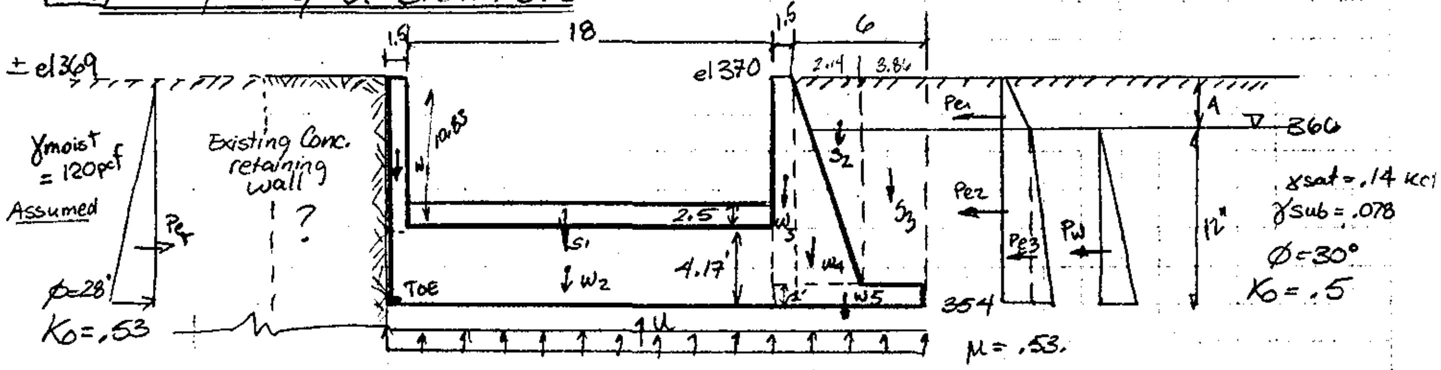
NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT: FORT FAIRFIELD - MAINE LPP
COMPUTATION: STOP LOG STRUCTURE
COMPUTED BY: ENestorides CHECKED BY: DATE: 2-9-87

I. Stop Log U-Channel:



The water line behind the battered U-channel wall is at el. 866. This elevation was used to roughly represent 60% of the unbalanced fill on both sides of the wall. The contribution of the existing Concrete wall on the U-channel will be modeled as the height of light fill it could retain.

Item	Computation	Vertical forces (K)	Horizontal forces (K)	Moment arm	Moment (K-Ft) (+)
W_1	(1.5)(10.83)(.15)	2.44		.75	1.83
W_2	(4.17)(19.5)(.15)	12.20		9.75	118.92
W_3	(1.5)(15)(.15)	3.38		20.25	68.45
W_4	½(15)(2.14)(.15)	2.41		21.71	52.32
W_5	(1)(7.5)(.15)	1.05		23.25	24.41
U	-12(.0625)(27)	-20.25		13.50	-273.38
S_1	2.5(18)(.12)	5.40		10.50	56.70
S_2	½(15)(2.14)(.14)	2.25		22.43	50.47
S_3	(3.86)(15)(.14)	8.11		25.07	203.32
P_{er}	$\frac{1}{2}(.53)(.12)(15)^2$		7.16	5.0	35.80
P_{e1}	$-\frac{1}{2}(.5)(.14)(4)^2$		-.56	13.33	-7.47
P_{e2}	-(.5)(.14)(4)(12)		-3.36	6.0	-20.16
P_{e3}	$-\frac{1}{2}(.5)(.078)(12)^2$		-2.81	4.0	-11.24
P_W	$-\frac{1}{2}(.0625)(12)^2$		-4.50	4.0	-18.00
Σ	—	ΣV = 16.99	ΣH = -4.07	—	ΣH = 281.97

Stability at toe:

- Overturning: $\frac{\Sigma M}{\Sigma V} = \frac{281.97}{16.99} = 16.60$ → within kern 100% > 75% in beari. OK

- Sliding: $SF = \frac{\Sigma V \tan\phi + et^{70}}{\Sigma H} \Rightarrow \frac{16.99(.53)}{4.07} \Rightarrow 2.21 > 1.33$ OK

- Bearing Pressure: $f_\pm = \frac{16.99}{27.0} \pm \frac{(16.99)(-3.1)(13.5)}{1640.25}$ $I = 1640.25$

$f_\pm = .63 \pm (-.43) \Rightarrow$ $f_+ = .20$ K/ft² , $f_- = 1.06$ K/ft² } < 4 K/ft² OK

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT: FORT FAIRFIELD - MAINE LPP
COMPUTATION: STOP LOG STRUCTURE / UPSTREAM
COMPUTED BY: ENestorides CHECKED BY: DATE: 2-9-87

StopLogs:

Top of rail ⇒ 361.5 $\qquad$ 367.5 - 361.5 ⇒ 6.0 ft. + 2 freeboard
Design flood ⇒ 367.5 $\qquad$ ⇒ 8.0

Max. Pressure ⇒ 8.0(.0625) ⇒ .50 K/ft²

Try 10"x10" ~ 9½" dressed $\qquad .50 \times \frac{9.5}{12} \Rightarrow .395$ K/ft

Opening is 18' wide ⇒ max. Moment = $\frac{wl^2}{8} = \frac{(.395)(18)^2}{8} \Rightarrow 16.03$ k·ft.

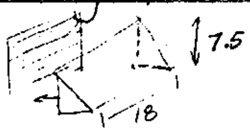
10" × 10": $S_{10x10} = 142.896\ in^3$ $\qquad \sigma = M/S = \frac{16.03 \times 12\ ^{in}/_{ft}}{142.896} \Rightarrow \sigma = 1.35\ K/in^2$

12" × 8": $S_{8x12} = 165.313$ $\qquad .5 \times \frac{7.5}{12} = .313 \qquad M = \frac{(.313)(18)^2}{8} = 12.66$ k.ft

$\sigma = \frac{12.66\,(12)}{165.313} \Rightarrow .92\ K/in^2$

σ = allowable (white, oak) 1200 psi ⟶ use 8"x12" section (12 required)

<u>Sliding of Structure due to Hydrostatic pressure</u>: flood 367.5, top of rail ⇒ 360.0 } ΔH=7.5



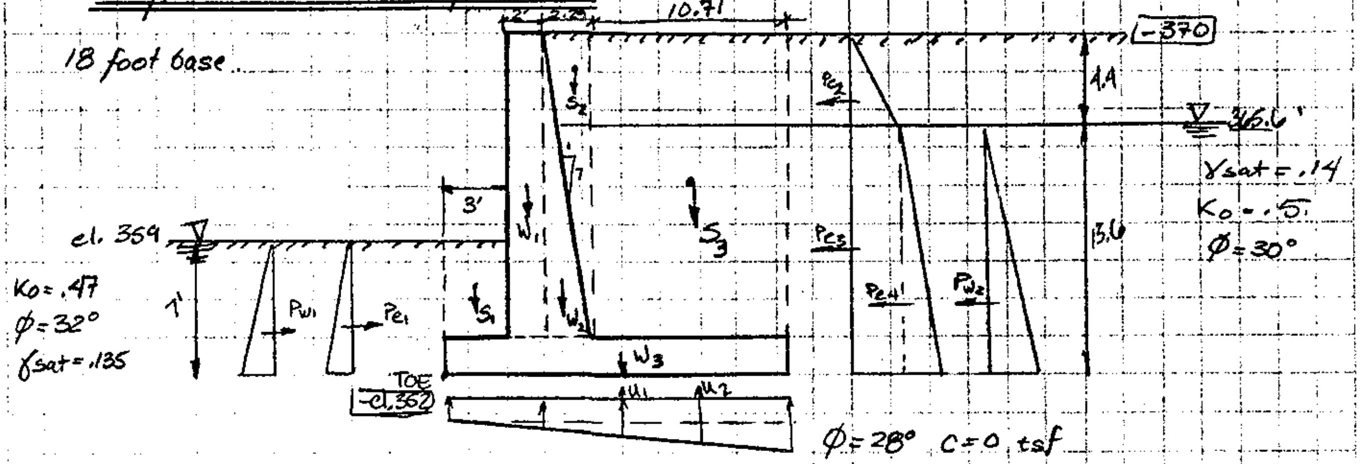
$\frac{1}{2}\ .0625(7.5)^2 \times 18 \Rightarrow 31.64\ \underline{K}$

Weight of structure ⇒ 9.08 K/ft × 16 ft ⇒ 145.28
including uplift

Sliding ⇒ $\frac{145.28(.53)}{31.64} \Rightarrow 2.43 > 1.33$ <u>OK</u> ✓

SUBJECT: Fort Fairfield Maine - LPP
COMPUTATION: StopLog Retaining Wall / Upstream
COMPUTED BY: E Nestorides CHECKED BY: DATE: 2/10/87

II. Typical Retaining Wall:



Item	COMPUTATIONS	Vertical forces	Horizontal forces	Moment arm	Moment
W_1	2(16)(.15)	4.8		4.0	19.2
W_2	½(16)(2.29)(.15)	2.75		5.76	15.84
W_3	2(18)(.15)	5.4		9.0	48.6
S_1	3(5)(.135)	2.03		1.5	3.05
S_2	½(16)(2.29)(.14)	2.56		6.53	16.71
S_3	10.71(16)(.14)	23.99		12.65	303.47
U_1	− 7(.0625)18	− 7.88		9.0	−70.88
U_2	− ½(.0625)(6.6)(18)	− 3.71		12.0	−44.56
Pe_1	½(.47)(.135)(7)²		.86	2.33	2.01
Pe_2	− ½(.5)(.14)(4.4)²		− .68	15.07	−10.21
Pe_3	− (.5)(.14)(4.4)(13.6)		− 4.19	6.8	−28.48
Pe_4	− ½(.5)(.078)(13.6)²		− 3.61	4.53	−16.34
Pw_1	½(.0625)(7)²		1.53	2.33	3.57
Pw_2	− ½(.0625)(13.6)²		− 5.78	4.53	−26.18
Σ	———	ΣV = 29.94	ΣH = −11.87	———	ΣM = 215.81

Stability at toe −352

- Overturning: $\frac{\Sigma M}{\Sigma V} = \frac{215.81}{29.94} \Rightarrow 7.2$ in mid ⅓ ⇒ 100% in bearing > 75% OK
- Sliding: $SF = \frac{\Sigma V \tan\phi + cL^{\to 0}}{\Sigma H} = 1.34 > 1.33$ OK
- Bearing Pressure: $f_\pm = \frac{\Sigma V}{area} \pm \frac{Mc}{I}$ $I = \frac{1(18)^3}{12} \Rightarrow 486\ ft^4$

$f_\pm = \frac{29.94}{18} \pm \frac{29.94(1.8)(9)}{486} \Rightarrow 1.66 \pm .998$

$f_+ = 2.66\ k/ft^2$
$f_- = 0.66\ k/ft^2$ } < 4 k/ft² OK

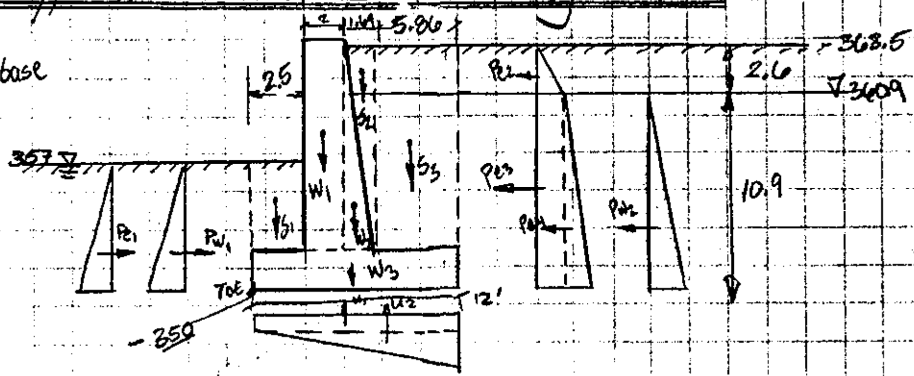
NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Fort Fairfield - Maine LPP
COMPUTATION Stop Log Retaining Wall / Upstream
COMPUTED BY E Nestorides CHECKED BY ______ DATE 2-10-87

III. Typical Section III: Retaining wall

12 ft base



Item	Computations	Vertical forces	Horizontal forces	Moment arm	Moment (+)
W_1	$2(11.5)(.15)$	3.45		3.5	12.08
W_2	$\frac{1}{2}(11.5)(1.64)(.15)$	1.41		5.05	7.12
W_3	$2(12)(.15)$	3.6		6.0	21.6
S_1	$(2.5)(5)(.135)$	1.69		1.25	2.11
S_2	$\frac{1}{2}(11.5)(1.64)(.14)$	1.32		5.59	7.38
S_3	$(5.86)(11.5)(.14)$	9.43		9.07	85.53
U_1	$-7(12)(.0625)$	-5.25		6.0	-31.5
U_2	$-\frac{1}{2}(3.9)(12)(.0625)$	-1.46		8.0	-11.68
P_{w1}	$\frac{1}{2}(.0625)(7)^2$		1.53	2.33	3.57
P_{w2}	$-\frac{1}{2}(.0625)(10.9)^2$		-3.71	3.63	-13.47
P_{e1}	$\frac{1}{2}(.47)(.075)(7)^2$		.86	2.33	2.00
P_{e2}	$-\frac{1}{2}(.5)(.14)(2.6)^2$		$-.24$	11.77	-2.82
P_{e3}	$-(.5)(.14)(2.6)(10.9)$		-1.98	5.45	-10.79
P_{e4}	$-\frac{1}{2}(.5)(.078)(10.9)^2$		-2.32	3.63	-8.42
Σ	—	$\Sigma V = 14.18$	$\Sigma H = 5.86$	—	$\Sigma M = 62.71$

Stability at Toe el. 360.0

- Overturning: $\frac{\Sigma M}{\Sigma V} = \frac{62.71}{14.18} = 4.42$ in mid. $\frac{1}{3}$ $\Rightarrow$ 100% bearing > 75% OK

- Sliding: $SF = \frac{14.18 \tan 28^\circ}{5.86} \Rightarrow 1.29 < 1.33$ close enough. OK

- Base Pressures: $f_{\pm} = \frac{14.18}{12} \pm \frac{(14.18)(1.58)(6)}{144} = 1.18 \pm .93$

$$\left.\begin{array}{l} f_+ = 2.11 \ ^k/_{ft^2} \\ f_- = .25 \ ^k/_{ft^2} \end{array}\right\} < 4 \ ^k/_{ft^2} \quad \text{OK}$$

SUBJECT: Fort Fairfield – Maine
COMPUTATION: Drainage pipe head-wall
COMPUTED BY: S. Gagnon   CHECKED BY: DATE: 2-13-87

DESIGN Method:

The headwall is designed and analyzed for stability as one unit. Criteria evolved from the Hydraulics appendix included:

1. Head wall soil elevation = 371
2. Side wall soil elevation ≈ 370.0 (estimate from topography map)
3. Pipe invert elevation ≈ 363.7 Min. (4' Ø Pipe)

The load case used represents a rapid drawdown situation which is an extreme loading for a retaining structure of this sort. Since the seismic zone is one for this area, the earthquake condition is not critical.

The structure is designed for a foundation of compacted gravel fill. Under the rapid draw back load case, the structure must satisfy the following stability criteria:

1. 75 % of the base must be in compression.
2. Factor of Safety against sliding must exceed 1.33.
3. Bearing pressures should not exceed the allowable 4 K/ft²

Design and Soil Parameters:

	γ_{sat}	γ_{sub}	ϕ	C
Compacted gravel Fill	.145	.0825	32°	0

- Soil Pressures using Ko ⇒ at rest coefficient.

$K_o = 1 - \sin\phi \quad \phi = 32° \quad K_o = .47$

- Sliding: $\mu = \tan\phi \quad \phi = 32° \quad \mu = .62$
  $C = 0$

- Rapid Drawdown: This condition was modeled by using 40% of the unbalanced fill as submerged and the remaining as saturated. The 40% is representable as a drainable fill.

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27 Sept 49

NEW ENGLAND DIVISION

CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Fort Fairfield - Maine

COMPUTATION Drainage pipe headwall

COMPUTED BY S. Guyn CHECKED BY DATE 2-13-87

The headwall is designed as both a retaining and outlet structure.

The structure is designed in accordance to the following Corps criteria:

1. EC 1110-2-510 "Working Draft of the Retaining and Flood wall Manual" 31 Aug. 1983 w/revisions 15 July 1985
2. ETL 1110-2-256 "Sliding stability for Concrete structures." 24 June 1981.

Soil and water pressures are developed from the following equations:

$$\text{Hydr. pressure} = \gamma_w h$$

$$\text{Saturated Soil Pressure} = \gamma_{sat}(h)(K_o)$$

$$\text{Submerged Soil Pressure} = \gamma_{sub}(h)(K_o)$$

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NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT

COMPUTATION

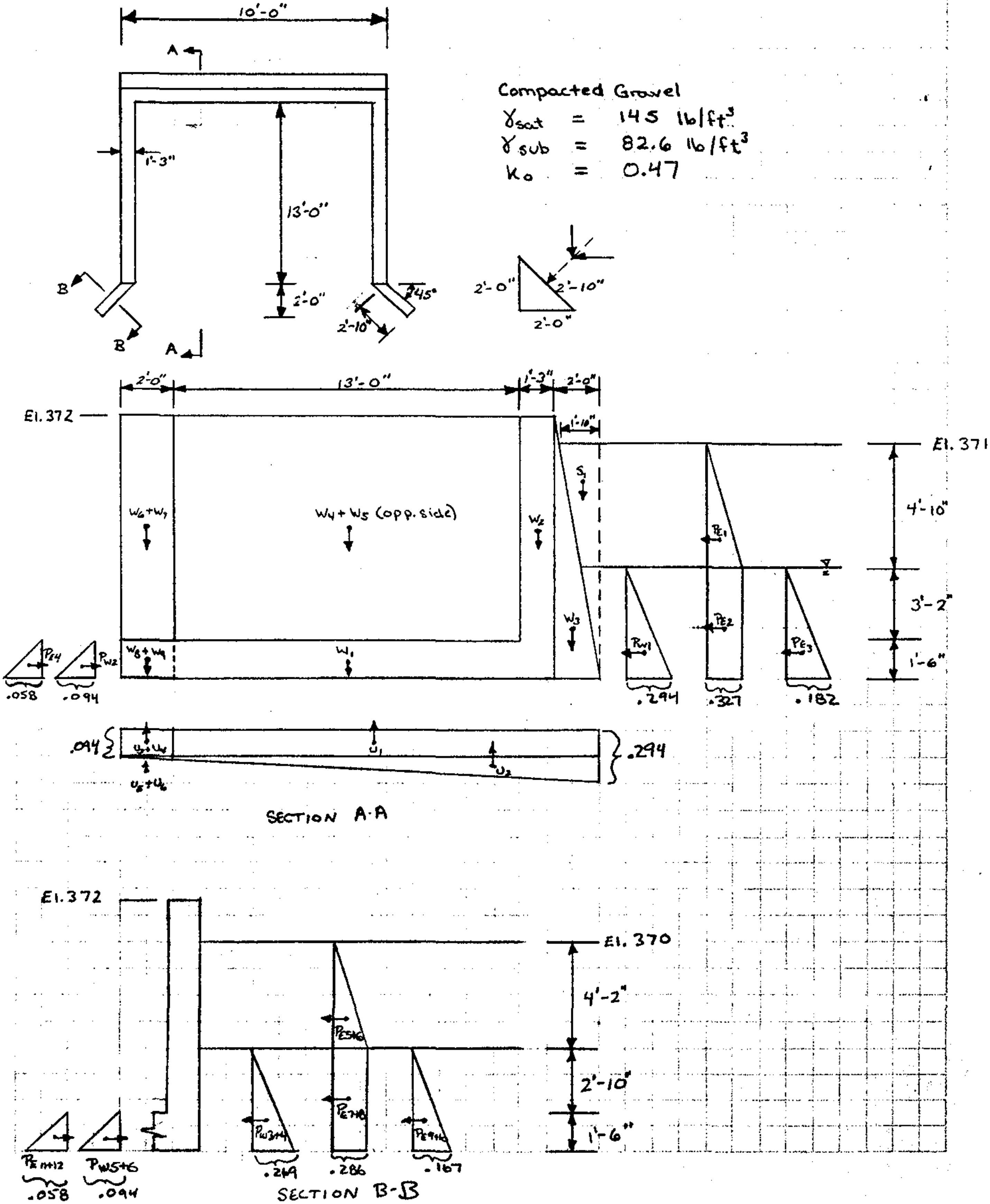
COMPUTED BY CHECKED BY DATE

Compacted Gravel

$\gamma_{sat} = 145$ lb/ft³

$\gamma_{sub} = 82.6$ lb/ft³

$K_o = 0.47$



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27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Fort Fairfield - Maine
COMPUTATION Drain-pip headwall
COMPUTED BY [signature] CHECKED BY DATE

ITEM	Computation	Vertical Forces (K)	Horizontal Forces (K)	Moment Arm (ft)	Moment @ TOE (K-ft)
W_1	(1.5)(15)(10)(.15)	+33.75		7.5	+253.12
W_2	(1.25)(10.5)(10)(.15)	+19.68		15.625	+307.5
W_3	1/2(2)(10.5)(10)(.15)	+15.75		16.92	+266.5
W_4+W_5	2(1.25)(13)(9)(.15)	+43.87		8.5	+372.89
W_5+W_6	2(2.83)(1.25)(9)(.15)	+9.55		1.0	+9.55
W_7+W_8	2(1/2)(2)(2)(1.5)(.15)	+0.9		0.667	+0.60
S_1	1/2(1.81)(9.5)(10)(.145)	+12.47		17.67	+220.34
U_1	(.094)(10)(18.25)	−17.15		9.125	−156.53
U_2	(1/2)(.2)(10)(18.25)	−18.25		12.167	−220.05
U_3+U_4	1/2(2)(.094)(2)(2)	−.376		1.0	−.376
U_5+U_6	1/2(.175)(2)(2)	−.350		1.33	−.465
P_{W1}	1/2(.294)(4.7)(10)		−6.9	1.567	−10.81
P_{W2}	1/2(.094)(1.5)(10)		+0.705	0.5	+0.352
$P_{W3}+P_{W4}$	2(1/2)(.269)(4.3)(2)		−2.31	1.43	−3.30
$P_{W5}+P_{W6}$	2(1/2)(.094)(1.5)(2)		+.282	0.5	+.141
P_{E1}	1/2(.327)(4.8)(10)		−7.85	6.3	−49.44
P_{E2}	(.327)(4.7)(10)		−15.37	2.35	−36.12
P_{E3}	(1/2)(.182)(4.7)(10)		−4.28	1.567	−6.70
P_{E4}	(1/2)(.058)(1.5)(10)		+.435	0.5	+.217
$P_{E5}+P_{E6}$	2(1/2)(.286)(4.2)(2)		−2.40	5.7	−13.69
$P_{E7}+P_{E8}$	2(4.3)(.286)(2)		−4.92	2.15	−10.57
$P_{E9}+P_{E10}$	2(1/2)(.167)(4.3)(2)		−1.44	1.43	−2.05
$P_{E11}+P_{E12}$	2(1/2)(.058)(1.5)(2)		+.174	0.5	+.087

$\Sigma V = 99.84$ $\Sigma H = 43.87$ $\Sigma M_{@TOE} = 921.98$

Stability @ TOE

– overturning: $\frac{\Sigma M}{\Sigma V} = \frac{921.98}{99.84} = 9.237 > \frac{18.25}{3} < 12.167$ ok

– Sliding: $SF = \frac{\Sigma V \tan\phi + cL^{\,0}}{\Sigma H} = \frac{(99.84)(.62)}{43.87} = 1.41 > 1.33$ ok

– Bearing Pressure: $f = \pm\frac{99.84}{154.0} \pm \frac{99.84(.105)(9.125)}{5065}$
(I = 5065)

$f\pm = .65 \pm .019 \Rightarrow f+ = .669,\ f- = .631$ } < 4.0 K/ft² ok

SUBJECT FORT FAIRFIELD - MAINE LPP
COMPUTATION PUMP STATION STABILITY
COMPUTED BY ENestorides CHECKED BY DATE 2/11/87

FORT FAIRFIELD - PUMP STATION

The pumping station was checked for stability along the bottom foundation depth of 346.3, in the north/south direction.

The structure was analyzed in accordance to the following Corps Criteria:

1. EC 1110-2-510: "Working draft of the Retaining and Flood Wall Manual" 31 August 1983 (w/Changes 15 July 8?

2. ETL 1110-2-256: "Sliding Stability of Concrete structures." 24 June 1981.

The soil surrounding the pumping station is at el. 360. The "worst" case loading involved the soil saturated all around the pumping station. Since fort Fairfield is in seismic zone One (minor damage), the earthquake load is not the critical loading.

The criteria for stability that must be satisfied are as follows:

a) That, the factor of safety against sliding is greater than 1.5 (this is an assumed value found acceptable for the given building and load case)
b) that the bearing pressures do not exceed the allowable $4\,k/ft^2$,
c) and, that 100% of the base be in compression.

Attached are sections upon which the weight of the structure was calculated.

Soil Parameters: Ko - at rest will be used for lateral soil pressures. The effects of the soil in the east/west direction resisting the overturning moment were not considered to be more conservative.

Assume: Compacted gravel fill around pumping station: $\gamma_{moist} = 135$, $\gamma_{sat} = 145$, $\phi = 35°$

$$K_o = 1 - \sin\phi = .43$$

Foundation Soil ⇝ GEB ⇒ $\phi = 28°$, $C = 0$ tsf $\mu = \tan\phi = .53$

allowable bearing capacity $= 4\,K/ft^2$

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NEW ENGLAND DIVISION

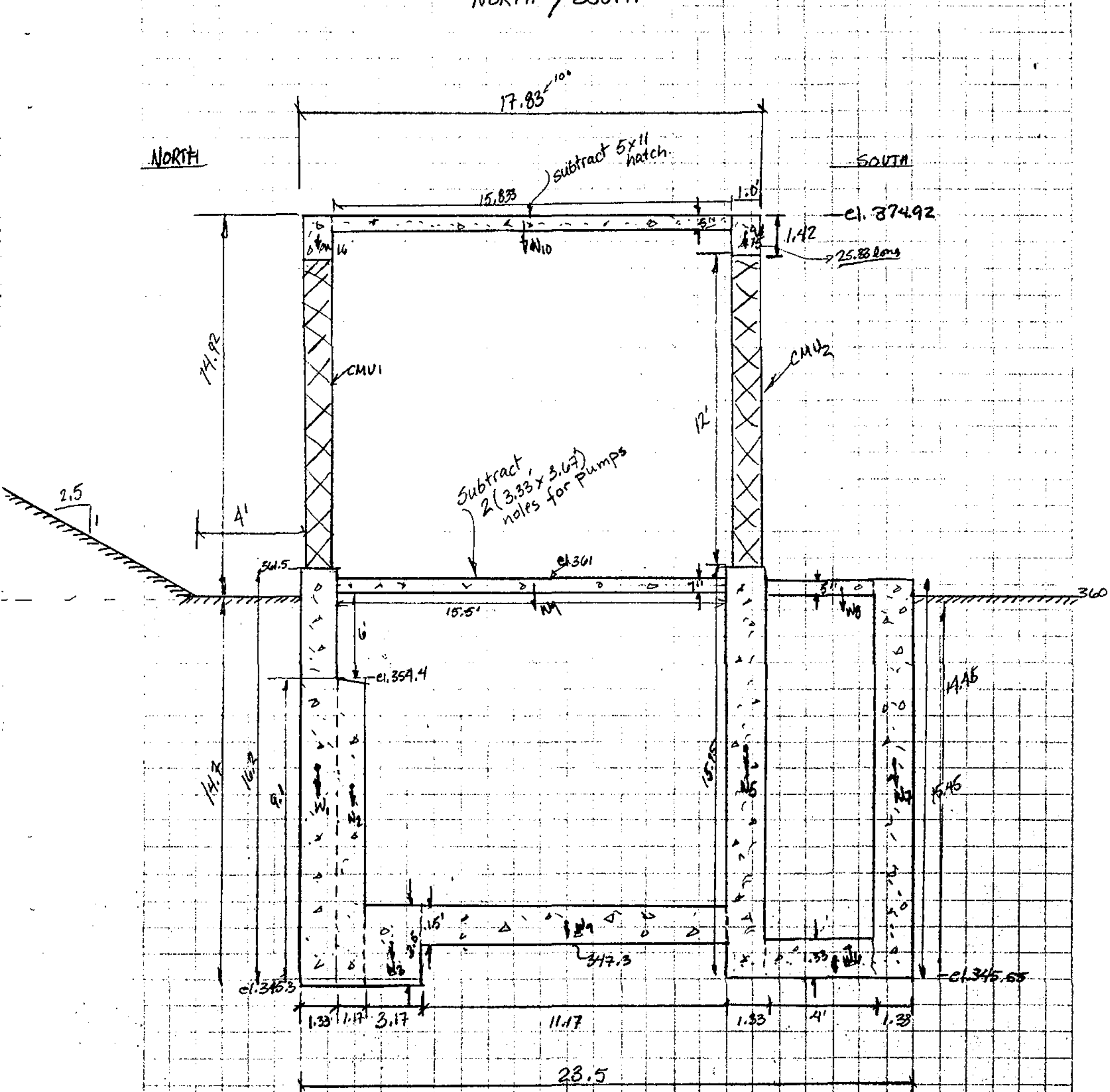
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Fort Fairfield - Maine LPP

COMPUTATION Pumping Station Stability

COMPUTED BY ENestorides CHECKED BY DATE 2-12-87

Pumping Station Typical Section.



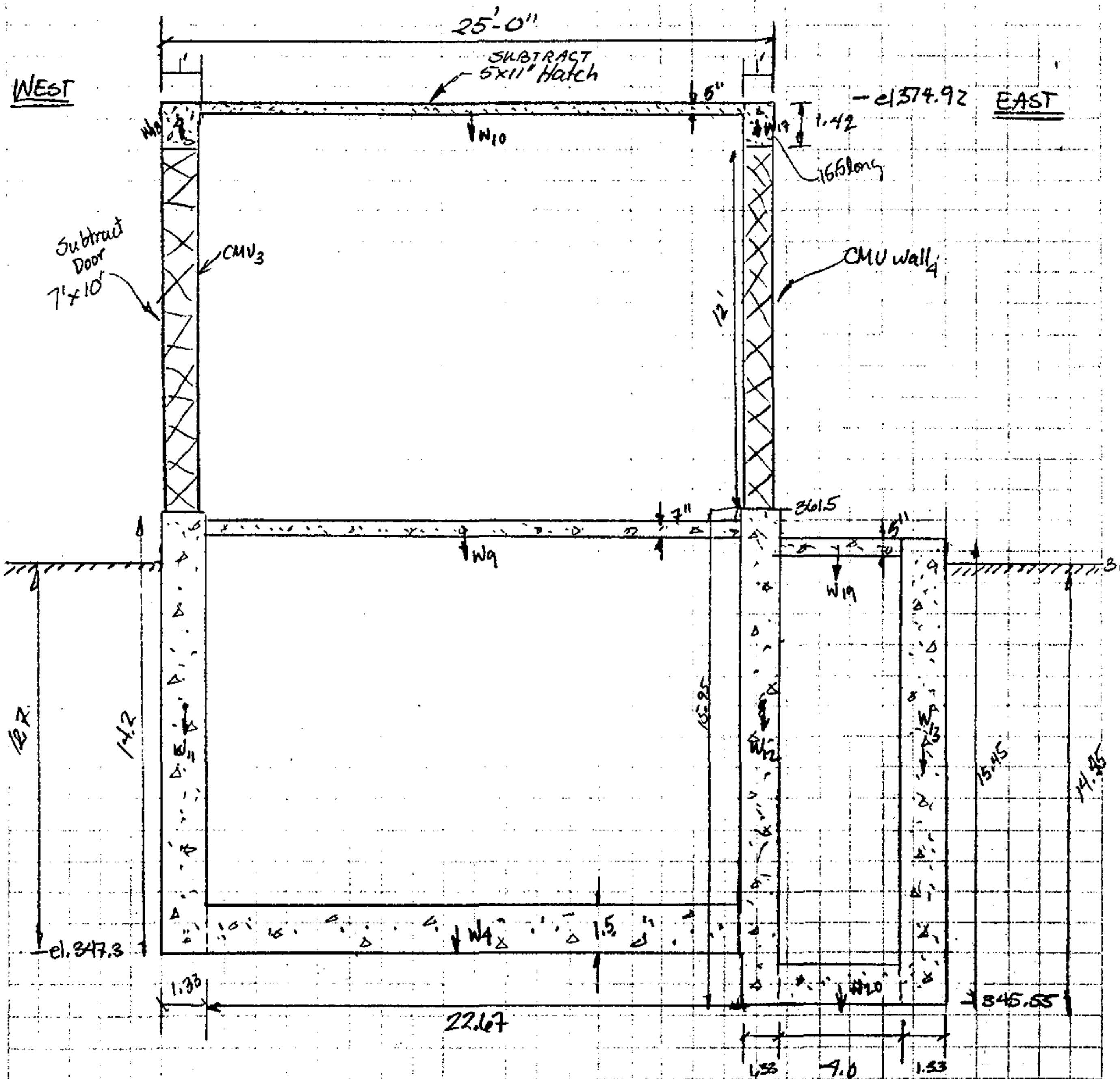
NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT: FORT FAIRFIELD - MAINE LPP

COMPUTATION: PUMPING STATION STABILITY

COMPUTED BY: ENestorides CHECKED BY: DATE: 2-12-87



ELEVATION

WEST / EAST

NOT TO ACCURATE SCALE

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NEW ENGLAND DIVISION
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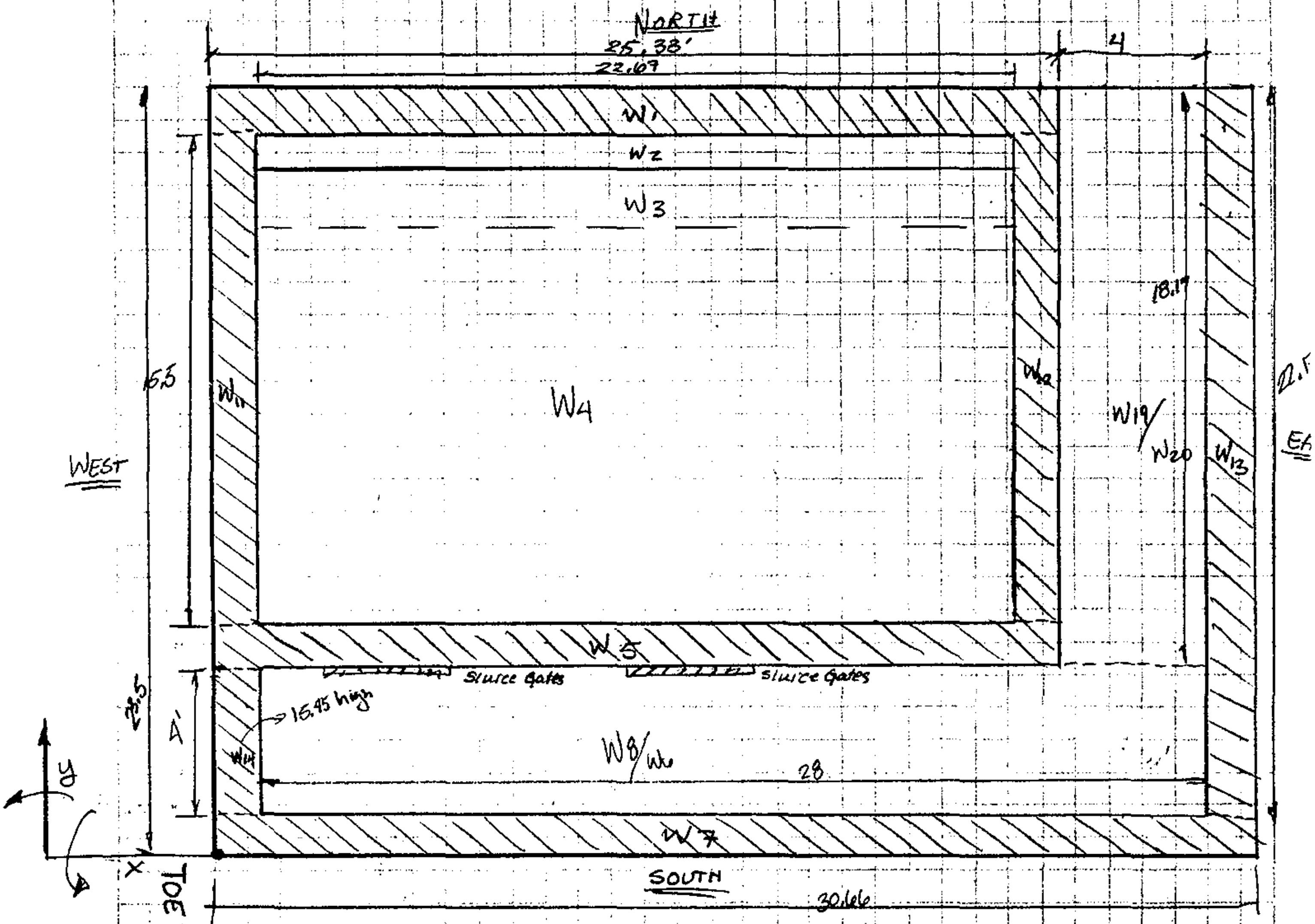
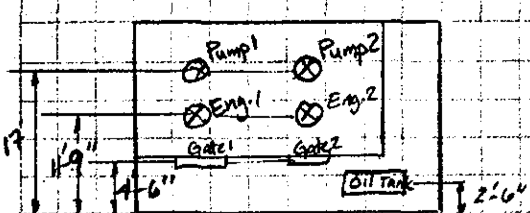
SUBJECT FORT FAIRFIELD - MAINE
COMPUTATION PUMPING STATION STABILITY
COMPUTED BY ENestorides CHECKED BY DATE 2/12-87



FOUNDATION PLAN

NOT TO ACCURATE SCALE

EQUIPMENT LAYOUT:



Pump ⇒ 3.0 k each
Engine ⇒ 3.3 k each
Gates ⇒ 1.3 k each (w/motor)
Oil Tank ⇒ 4 k

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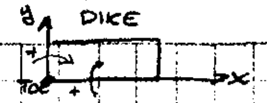
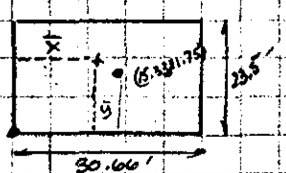
NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT: FORT FAIRFIELD - MAINE LPP
COMPUTATION: Pumping Station Stability
COMPUTED BY: ENestorides CHECKED BY: ______ DATE: 2-12-87



WEIGHT OF STRUCTURE AND CENTER OF GRAVITY

Item	COMPUTATION	WEIGHT (K)	$\bar{y}$ Moment arm	Mx Moment about x-axis	$\bar{x}$ moment arm	My Moment (k-f) arm
Concrete						
W_1	(1.33)(16.2)(25.33)(.15)	81.86	22.84	1869.77	12.67	1037.22
W_2	(1.17)(9.1)(22.67)(.15)	36.21	21.59	781.67	12.67	458.72
W_3	(3.17)(3.5)(22.67)(.15)	37.73	19.42	732.69	12.67	478.02
W_4	(11.17)(1.5)(22.67)(.15)	56.98	12.25	697.95	12.67	721.94
W_5	(1.33)(15.95)(25.33)(.15)	80.60	6.0	483.60	12.67	1021.21
W_6	(4.0)(1.33)(28)(.15)	22.34	3.33	74.41	15.33	342.53
W_7	(1.33)(15.45)(30.66)(.15)	94.50	.67	63.32	15.33	1448.72
W_8	(4.0)(5/12)(28)(.15)	7.06	3.33	23.50	15.33	108.17
W_9	(15.5)(7/12 .58)(22.67)(.15) - 2(3.33)(3.67)(.15)(.58)	28.44	14.42	410.16	12.67	360.39
W_{10}	(15.83)(.42)(23.0)(.15) - (5)(11)(.15)(.42)	19.47	14.59	284.11	12.50	243.41
W_{11}	(1.33)(14.2)(15.5)(.15)	43.91	14.42	633.18	.67	29.42
W_{12}	(1.33)(15.95)(15.5)(.15)	49.32	14.42	711.21	24.66	1216.27
W_{13}	(1.33)(15.45)(22.17)(.15)	68.33	12.42	848.71	29.99	2049.34
W_{14}	(4.0)(15.45)(1.33)(.15)	12.33	3.33	41.06	.67	8.26
W_{15}	(1.0)(1.42)(25.33)(.15)	5.40	6.17	33.29	12.67	68.36
W_{16}	(1.0)(1.42)(25.33)(.15)	5.40	23.0	124.09	12.67	68.36
W_{17}	(1.42)(1.0)(15.5)(.15)	3.30	14.42	47.61	24.33	80.33
W_{18}	(1.42)(1.0)(15.5)(.15)	3.30	14.42	47.61	.67	2.21
W_{19}	(4.0)(.142)(18.19)(.15)	4.58	14.42	66.10	27.33	125.28
W_{20}	(4.0)(1.33)(18.19)(.15)	14.52	14.42	209.32	27.33	396.91
CMU Walls						
CMU1	(12)(1)(25.33)(.08)	24.32	22.84	555.40	12.67	308.09
CMU2	(12)(1)(25.33)(.08)	24.32	5.84	142.01	12.67	308.09
CMU3	[(12)(1)(15.5) - (1)(7)(10)](.08)	9.28	14.42	133.82	.67	6.22
CMU4	(12)(1)(15.5)(.08)	14.88	14.42	214.57	24.66	366.94
Σ	—	W_s = 748.38	—	ΣM_x = 9,229.16	—	ΣM_y = 11,254.21



$$\bar{x} = \frac{\Sigma M_y}{W_s} = \frac{11254.21}{748.38} \qquad \bar{x} = 15.04 \text{ ft}$$

$$\bar{y} = \frac{\Sigma M_x}{W_s} = \frac{9229.16}{748.38} \qquad \bar{y} = 12.33 \text{ ft}$$

$$I_x \Rightarrow \frac{(30.66)(23.5)^3}{12} = 33158 \text{ ft}^4 \qquad \text{Area} \Rightarrow (23.5)(30.66) \Rightarrow 720.51$$

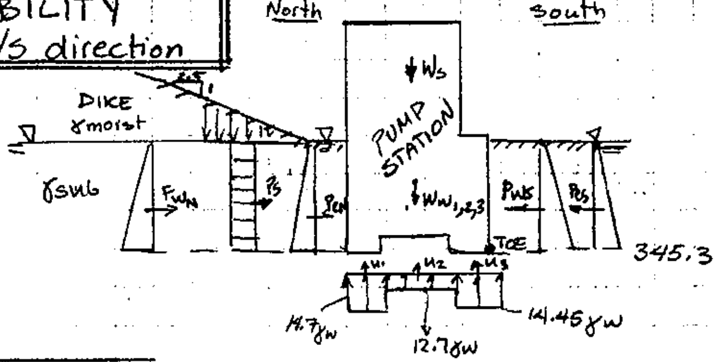
$$I_y = \frac{(23.5)(30.66)^3}{12} = 56442 \text{ ft}^4$$

SUBJECT Fort Fairfield - Maine LPP

COMPUTATION Pumping Station Stability

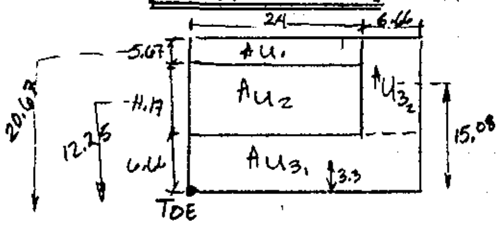
COMPUTED BY ENestorides CHECKED BY DATE 2-12-87

STABILITY N/S direction



LOAD CASE 1: Flood Conditions, surrounding backfill submerg. Water in sump chamber.

1. UPLIFT:



Uplift felt on base slab.

$A_{U1} \Rightarrow 136.08$ $\quad \uparrow U_1 = (136.08)[14.7(.0625)] = \underline{125.02\ k}$

$A_{U2} \Rightarrow 268.08$ $\quad \uparrow U_2 = (268.08)[12.7(.0625)] = \underline{212.79\ k}$

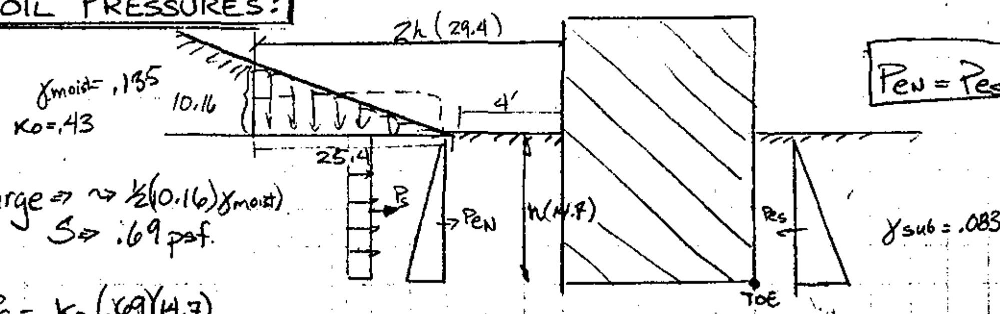
$A_{U3} \Rightarrow 316.35$ $\quad \uparrow U_3 \Rightarrow (316.35)[(14.45)(.0625)] = 285.79\ k$

$\uparrow U_{31} = \underline{184.42\ k}$

$\uparrow U_{32} = \underline{101.29\ k}$

- Horizontal Hydrostatic Forces Cancel Out.

2. SOIL PRESSURES:



$P_{EN} = P_{ES}$ and Cancel out

Surcharge ⇒ ~ ½(0.16)(γmoist)

S ⇒ .69 psf.

$P_s = K_o(.69)(14.7)$

$\Rightarrow 4.33\ k/f$ Linear.

Resultant $P_s \Rightarrow 4.33\ k/ft \times 30.66 = \underline{132.91\ k}$ at 7.35 ft above to

3. WATER IN SUMP CHAMBER:

Area of sump chamber: (15.5 ft.)(22.67) = 351.39 ft²

water at el. 357: (357 - 348.8) ⇒ 8.2 ft of water

Volume of Water in sump chamber ⇒ 2,881.40 ft³

$W_{W1} = (.0625)(2881.40) \Rightarrow \underline{180.09\ k}$ at 14.42 ft from toe.

3' of water in By-Pass Channel: $Vol_I \Rightarrow 4(28)(3) \Rightarrow 336\ ft^3$

$W_{W2} \Rightarrow (336\ ft^3)(.0625\ k/ft^3) = \underline{21\ k}$ at 3.33 ft from

$Vol_{II} = 4(18.19)(3) \Rightarrow 218.28$

$W_{W3} = (218.3)(.0625) = \underline{13.64\ k}$ at 14.41 ft from toe

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

PAGE 25

SUBJECT: FORT FAIRFIELD - MAINE LPP
COMPUTATION: PUMPING STATION STABILITY
COMPUTED BY: ENestorides CHECKED BY: DATE: 2-12-87

Stability Analysis [with equipment as shown in layout on previous pages, no other extras (doors, louvers, hatch etc.)]

Item	VALUE (K)	Moment arm ft	Moment (K.ft)
W_s	748.38	12.33	9227.53
W_{w_1}	180.09	14.42	2596.90
W_{w_2}	21	3.33	69.93
W_{w_3}	13.64	14.41	196.55
U_1	- 125.02	21.67	- 2709.18
U_2	- 212.79	12.25	- 2606.68
U_{S_1}	- 184.42	3.33	- 614.12
U_{S_2}	- 101.29	15.08	- 1527.45
* P_S	- 132.91	7.35	- 976.89
Pump 1	3.0 K	17.0	51.0
Pump 2	3.0 K	17.0	51.0
Eng 1	3.3 K	11.75	38.78
Eng 2	3.3 K	11.75	38.78
Gate 1	1.3	4.5	5.85
Gate 2	1.3 k	4.5	5.85
oil tank	4.0 k	2.5	10.0
Σ	ΣV = 358.79 ΣH = 132.91		ΣM = 3857.62

* Horizontal force.

Stability at Toe el. 345.3

Overturning: $\frac{\Sigma M}{\Sigma V} = \frac{3857.62}{358.79}$

$\Rightarrow 10.75$ ft.

$7.8 < \text{Mid } 1/3 < 15.7$ } OK within Mid ⅓ 100% in bearing

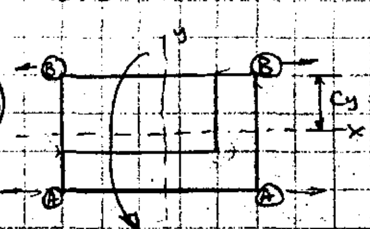
Sliding:

$SF = \frac{\Sigma V \tan\phi + cL}{\Sigma H} = \frac{358.79 (\tan 28}{132.91}$

$SF = 1.44 < 1.5$ OK

This is a bit low however given that the effects of the soil press. resisting sliding in the East/West direction were not considered, the above factor is found acceptable.

Bearing Pressure: $f_\pm = \Sigma V/\text{area} \pm \frac{M_x c_y}{I_x}$ (N/S direction)



$I_x = 33{,}158 \text{ ft}^4$ $c_y = 11.75$ ft Area = 720.51

$f_\pm \Rightarrow \frac{358.79}{720.51} \pm \frac{358.79(1)(11.75)}{33{,}158} \Rightarrow .498 \pm .127$

along (A-A) $f_+ = .63 \text{ K/ft}^2$
along (B-B) $f_- = .37 \text{ K/ft}^2$ } $< 4^k/\text{ft}^2$ OK.

SECTION D

SOCIAL AND ECONOMIC ANALYSIS

FORT FAIRFIELD, ME
ECONOMIC ANALYSIS

Table of Contents	Page No.

Introduction

The purpose of this section is to measure the beneficial contributions to national economic development that are associated with the water resources improvement plans for the Fort Fairfield floodplain. The extent to which the flood control needs of the area are met by the plans will be determined by estimating the dollar value of inundation reduction benefits produced by the plans. Explanatory rationale and supporting documentation will be presented. The measure of each plan's economic justification is the benefit-cost ratio, which is calculated by dividing the dollar value of the total annual benefits to be realized over the plan's economic life by the annual charges for the plan's total cost. A benefit-cost ratio of 1.0 or greater is necessary for Federal participation in water resources improvement projects. Simply, one dollar's worth or more of flood reduction benefits is required for each dollar to be expended on project construction. If more than one plan of improvement has a benefit-cost ratio greater than 1.0 then the plan with the greatest amount of net benefits (ie. total annual benefits minus total annual costs) is chosen. The plan which maximizes net benefits allocates limited resources in the most efficient manner and provides the greatest return on public investment. The analysis contained in this section was performed in accordance with Economic Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies, Water Resources Council, 1983. Dollar values stated in this section reflect the December 1986 price level. Discounting and amortization was performed at 8-7/8 percent, the current interest rate for Federal water resources improvement project evaluation.

Socio-Economic Setting

The town of Fort Fairfield is located in Aroostook County, Maine. This county contains more than 20 percent of Maine's land area but only 8 percent of its people. The rural nature of the county is indicated by its population density per square mile of 13.6 compared to 35.3 statewide. The population of Fort Fairfield is 4,376 (1980 U.S. Census). Both the town and Aroostook County have experienced population declines over the past 20 years, while the state of Maine population overall has been growing since 1940.

TABLE 1
POPULATION TRENDS 1960 - 1980

	1960	1970	1980	% Change 1960-1970	% Change 1970-1980
Fort Fairfield	5,876	4,859	4,376	-17.3	-9.9
Aroostook County	106,064	94,078	91,331	-11.3	-2.9
State of Maine	969,300	993,700	1,124,660	+2.5	+13.2

The population declines in Fort Fairfield and Aroostook County can be traced to fewer agricultural jobs due to mechanization and a decline in competitive market position in the potato industry. Other employment sectors are also not providing job opportunities in sufficient numbers to halt emigration of job-seekers from the county. The increases in population for the state of Maine reflect the growth and development in the southern counties, especially the seacoast communities and those nearby.

The economic well-being of Fort Fairfield inhabitants can be measured by examining per capita income, median family income and percentage of families at or below the poverty level.

TABLE 2
INDICATORS OF ECONOMIC WELL-BEING

	Per Capita Income	Median Family Income	% of Families below Poverty Level
Fort Fairfield	$4,460	$14,022	10.2%
Aroostook County	$4,826	$13,924	13.3%
State of Maine	$5,768	$16,167	9.8%

Fort Fairfield and Aroostook County obviously have income measures below statewide figures because of their rural nature and lack of a strong industrial base. However, Fort Fairfield families do fare slightly better than average county families in terms of median income. Also, while the town's poverty level percentage is nearly that of the state it is 3 percentage points lower than that of Aroostook County.

According to the 1980 U.S. Census, the Fort Fairfield labor force was 1,735 people of which 1,598 were employed. Of the employing industries, services accounted for the largest share (23%) mostly in health and education. Other major employing industries are: manufacturing (17.5%), agriculture (13.5%), retail trade (13.7%), public administration (7.7%) and construction (6.3%). The majority of employed persons are private wage and salary workers (63%) with the remainder working for Federal, state or local government (26%) or self-employed (11%).

Study Area

The actual study area is comprised of approximately 25 acres along both sides of Main Street in the commercial district of Fort Fairfield. Main Street is located adjacent to the Aroostook River and its low-lying one-half mile stretch between Peterson's Garage and the Canadian Pacific Railroad Office has been the scene of many floods. Most of the flooding occurs during the springtime because of snowmelt and in many instances is exacerbated by ice jams.

The character of the study area is mostly commercial, however, there is a concentration of senior citizen housing units. There are 30 commercial structures in the area which house 41 separate commercial activities, 4 fraternal organizations and one government agency. Of these 30 structures, 5 have apartments on the second story and one has a total of 25 apartments on its second and third stories. There is only one traditional two-story, two-family house in the study area, but there are two senior citizen housing complexes. The first, Northern House, is a 3-story structure which contains 26 apartments. The second is the Fields Lane Senior Citizen Complex and is operated by the Housing Authority of Fort Fairfield. The complex is a campus type layout with 9 detached structures accounting for a total of 40 units plus a community center. Rounding out the structural inventory of the study area are two government buildings, one the U.S. Post Office and the other the Fort Fairfield Municipal Building which is occupied by town offices, the Police Department and Fire Department.

Valuation of Properties in the Study Area

In November 1986 the Town of Fort Fairfield provided the total value, based on Town Assessor's records, of the properties in the Main Street study area. The value of land is $502,860 and buildings is $3,641,000 for a total value of $4,143,860. Town officials indicate that this figure is roughly 94 percent of current market value.

Flood Damage Surveys

Flood damage surveys are performed at the start of every Corps of Engineers flood control study in order to determine the need for improvements by estimating the magnitude of potential flood-related losses. These losses are estimated, at each flood-prone structure and site, starting at the elevation at which flooding and damage begins up to the elevation of floodwater associated with a very rare event such as the 500 year storm. Damages are estimated in one-foot increments between these two limits. The categories of these losses are: commercial, industrial, residential, agricultural and public. The two types of losses are physical and non-physical. Physical losses relate to grounds, site, structure, contents, utilities and clean-up. Non-physical losses are those additional induced costs which result from loss of use of a flooded structure. Residential non-physical losses are the costs of food, lodging and necessities while unable to use one's residence. For commercial and industrial firms non-physical losses are measures such as lost income and profit while shut down plus the cost of temporary quarters and services. In addition to the structure-related loss categories above, the flood damage survey estimation process also covers two general loss categories: (i) cost of emergency services and (ii) damages and costs to transportation, communication and utility systems.

The first flood damage survey of the Fort Fairfield study area was performed in October 1977 by a private consulting engineering firm as part of the larger St. John River Basin study. In October 1982, damage evaluators from the New England Division performed a major on-site update. Updates have been performed recently in November 1985 and December 1986 to document improvements which have taken place in the study area.

Recent and Planned Improvements in Study Area

In 1985 the State of Maine awarded a Community Development Block Grant in the amount of $820,000 to fund the 2-year Fort Fairfield Downtown Revitilization Project. Under this project certain commercial buildings were renovated and expanded and some older buildings were razed. Private investment in the study area was also made during 1986. The Irving Oil Co. constructed a large gas station, grocery store and liquor store. In 1987, the State of Maine, Department of Transportation plans to completely excavate and construct a new roadway and sidewalks for Main Street in the study area. Other improvements for Main Street scheduled for 1987 are: (i) the installation of 125 new street lights, (ii) installation of a new 8 inch sanitary sewer line (1600 linear feet) with manholes and service extensions and (iii) reinforcement of the existing telephone system, both underground and aerial, along Main Street by New England Telephone. The total cost for these 4 scheduled improvements is $1,500,000.

Susceptibility to Flooding

One indicator of an area's susceptibility to damage from flooding is the relationship of the first floor elevation of structures in the floodplain to the elevation of floodwaters from certain events. First floor elevations were obtained for all floodplain structures by a field survey crew and potential flood elevations were obtained from an "elevation vs. frequency" curve produced by the Water Control Branch (Hydrologic Engineering Section) of the New England Division. The summary table below shows the relationship between flood elevation, frequency and number of structures affected. The salient point of the table is that even a storm of 10 year frequency will produce a flood level that will cover the first floor of 25 of the 43 floodplain structures.

TABLE 1
STRUCTURES SUSCEPTIBLE TO FIRST-FLOOR FLOODING
FORT FAIRFIELD STUDY AREA

Event (year)	Annual % Chance of Occurrence	Flood Elevation (NGVD)	Structures w/ First Floor Flooding: Number	Structures w/ First Floor Flooding: % of Total
100 yr.	1%	367.3'	37	86%
50 yr.	2%	366.4'	33	77%
10 yr.	10%	363.9'	25	58%

Recurring Losses

Recurring losses are those potential flood related losses which are expected to occur at various stages of flooding under present day development conditions. Table 2 below displays the dollar value of potential flood-related losses, by damage category, that are estimated to occur if that specific flooding event were to occur today.

TABLE 2
RECURRING FLOOD LOSSES
FORT FAIRFIELD STUDY AREA

Category	10 Year Event (el. 363.9')	50 Year Event (el. 366.4')	100 Year Event (el. 367.3')	500 Year Event (el. 369.2')
Properties	$1,107,000	$3,592,000	$4,678,000	$6,795,000
Emergency Costs	14,800	24,600	33,800	53,200
Downtown Roads	20,000	239,400	273,100	273,100
Railroads	87,300	174,500	174,500	174,500
Total Losses	$1,229,100	$4,030,500	$5,159,400	$7,295,800

Annual Losses

Recurring losses, discussed above, are informative inasmuch as they relate the dollar value of flood losses to specific depths of flooding, however they don't offer any information as to what the chances are of those flooding depths occurring in any given year. For the purpose of determining the severity of potential flooding the statistical concept of expected value is employed. For flood control studies the term used to measure the severity of potential flooding on an annual basis is "annual losses." Annual losses are calculated by integrating two sets of data: (i) recurring losses displayed in one-foot increments of flood depth from start of damage to the 500 year storm elevation and (ii) the estimated annual percent chance that flooding will reach each specific elevation for which recurring losses were estimated. Recurring losses are obtained by the flood damage survey process and the annual percent chance of occurrence for each event is obtained form a stage-frequency curve. This curve, estimated by the Hydrologic Engineering Section at NED, displays flood stages on the X-axis and the annual percent chance of reaching that stage on the Y-axis. Annual losses are computed for each event from the one that first causes damage to the 500 year event. Losses for all events are aggregated and this total estimate of expected annual losses represents the degree of flooding severity in the study area. The effectiveness of each alternative plan that is formulated for flood reduction is measured by the extent to which it reduces annual losses. Annual losses, by category, for the Fort Fairfield study area are displayed in Table 3.

TABLE 3
ANNUAL LOSSES
FORT FAIRFIELD STUDY AREA

Category	Annual Losses
Properties	$398,400
Emergency Costs	2,800
Downtown Roads	12,300
Railroads	47,000
Total	$460,500

Economic Benefit Analysis

Benefits from plans for reducing flood hazards accrue primarily through the reduction in actual or potential damages associated with land use. Benefits fall into three categories reflecting different responses to a flood hazard reduction plan. The inundation reduction benefit accrues when land use is the same with or without the plan and is defined as the increased net income generated by that use. The intensification benefit also accrues when land use is unchanged and is defined as the increase in net income based on a modification of the method of operation by floodplain occupants because of the plan. The location benefit accrues when an activity is added to the floodplain because of a plan and is measured as the difference between aggregate net incomes in the economically affected area with and without the plan.

Under the "with plan" condition for the Fort Fairfield study area, land use is projected to remain essentially the same. Since the area is the center of commercial activity and has a considerable number of permanent elderly housing units, it is projected that these functions will continue into the foreseeable future. This projection is nearly irrefutable based on the public and private investments in the area's infrastructure and commercial activities during 1985 to 1987. There probably will be modifications to existing activities and development on some of the few vacant lots, with the plan, but it is not expected to be on a large enough scale to significantly affect future losses and benefits. Therefore, benefits which accrue to the improvement plans will be measured under the category of inundation reduction only.

Inundation Reduction Benefit

The increase in net income that accrues under this category is measured by the decrease in the dollar value of outlays associated with reduced flood losses. The national economic development (NED) objective is satisfied if an improvement plan produces the beneficial impact of reducing annual losses.

Improvement Plans Evaluated

Three improvement plans, each offering a different level of protection, were evaluated. All three plans involve a 3000 foot long earthen dike which would extend from just upstream of Peterson's Repair Garage downstream to the Canadian Pacific Railroad Office. The plans to be evaluated offer flood protection against the following 3 events: (i) 500 year, (ii) 100 year and (iii) 50 year.

Benefit Estimation

Benefits for inundation reduction were calculated based on the flood elevation corresponding to each event. The top elevation of each dike plan is that flood elevation plus and additional 3 feet of freeboard to account for wave run-up and wind effects. Corps of Engineers regulations allow benefits to be taken up to the top of the dike plus 50 percent (1.5 feet) of the freeboard range. The benefits to each plan are the summation of annual losses prevented by the dike taken to an elevation 1.5 feet below the absolute top of dike including freeboard. The benefits for each plan are enumerated in Table 4.

TABLE 4
ANNUAL BENEFITS - INUNDATION REDUCTION
FORT FAIRFIELD STUDY AREA

	Annual Inundation Reduction Benefits		
	Level of Protection		
Category	500 Year (el. 369.5')	100 Year (el. 368')	50 Year (el. 367')
Properties	$387,400	$362,200	$327,000
Emergency Costs	2,800	2,600	2,300
Downtown Roads	11,900	10,700	8,900
Railroads	46,600	46,000	44,800
Total	$448,700	$421,500	$383,700

Reduced Pumping Costs

A second type of flood related cost that will be reduced by the dike plan is the increased pumping costs at the Fort Fairfield Sewage Treatment Plant during times of flooding. There is a sewer pipe which runs along the entire length of the site where the dike would be constructed. This pipe would require relocation closer to Main Street, away from the river bank if the dike were to be constructed. In order to determine if economic benefits would accrue to this relocation, the manager of the Fort Fairfield Utilities District was interviewed. The pipe does not currently sustain direct damage from flooding or erosion. It was installed in 1976, is made of PVC, is buried 13 to 17 feet below ground and has an expected life of 60 years. However, during periods of flooding at the pipe's location, especially in springtime, inflow and infiltration of floodwaters into the pipe occurs at manholes and around some pipe joints. Pumping at the treatment plant increases dramatically from an average of 0.4 MGD to 1.5 MGD during times when floodwaters enter the system and continues at the elevated rate for 2 weeks after flooding subsides. There are two negative effects caused by this inflow. First, the pumping system is overburdened and must pump flood water that doesn't need treatment. Because of this, untreated sewage also gets pumped into the river. The Utilities District is currently under a consent decree from the Maine Department of Environmental Protection to control the inflow. Secondly, the increased volume which needs to be pumped during times of flooding increases the pumping costs. Under the with-plan condition, the section of pipe where inflow and infiltration occurs will be relocated to the inside of the dike, closer to Main Street and further away from the riverbank. The manger of the Utilities District indicates that this relocation of the pipe should solve the inflow/infiltration problem as the manholes will be in the flood protection area. The pumping plant will not be overburdened, pumping costs will remain at normal levels, and untreated sewage will not be pumped into the river, thereby keeping the Utility District in compliance with its State and Federal licenses. The benefit to be realized with the project is estimated to be $2,000 annually in reduced pumping and associated repair costs.

Reduction in Flood Insurance Overhead Costs

A cost of floodplain occupancy is flood insurance overhead costs. This administrative cost is national in nature and will be eliminated with the 500 year and 100 year dike improvement plans. The 1986 overhead cost per policy is $67 and an estimated 36 policies are in effect in the study area. With the improvement plan the annual benefit is $2,400.

Summary of Benefits

The annual benefits expected to accrue under each of the 3 flood protection plans are exhibited in Table 5 below.

TABLE 5
SUMMARY OF ECONOMIC BENEFITS
FORT FAIRFIELD FLOOD REDUCTION PLANS

Category	Annual Benefits 500 Year Protection	100 Year Protection	50 Year Protection
Inundation Reduction:			
Properties	$387,400	$362,200	$327,700
Emergency Costs	2,800	2,600	2,300
Downtown Roads	11,900	10,700	8,900
Railroads	46,600	46,000	44,800
Reduced Pumping Costs (Sewage Treatment Plant)	2,000	2,000	2,000
Reduction in Flood Insurance Overhead Costs	2,400	2,400	-
TOTAL BENEFITS	$453,100	$425,900	$385,700

Economic Justification

The ultimate purpose of the economic analysis is to compare the benefits estimated for each plan to the annual costs of plan implementation in order to determine the benefit-cost ratio which is the measure of economic justification and indicator of Federal participation.

TABLE 6
ECONOMIC EVALUATION OF PLANS

	500 Year Protection	100 Year Protection	50 Year Protection
Total Annual Benefits	$453,100	$425,900	$385,700
Total Annual Costs			
Benefit-Cost Ratio			
Net Benefits			

SECTION E

REAL ESTATE